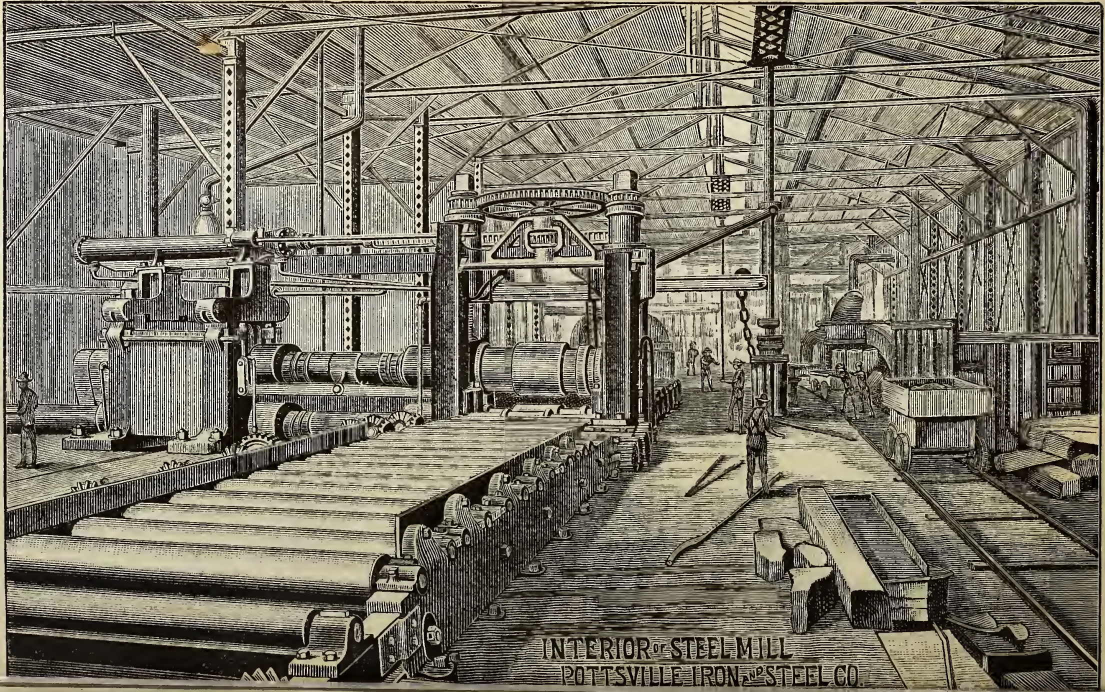
POTTSVILLE IRON AND STEEL CO.

Pottsville, Penna.

E A Henderson
P I F



INTERIOR OF STEEL MILL
POTTSVILLE IRON AND STEEL CO.

#70

HANDBOOK

FOR

Engineers, Architects, and other Workers

IN

IRON AND STEEL,

CONTAINING

TABLES OF CAPACITY OF I BEAMS AND CHANNELS
OF IRON AND STEEL,

AS MANUFACTURED BY THE

POTTSVILLE IRON AND STEEL CO.

OF POTTSVILLE, PENNA.

ALSO,

DESIGN AND CALCULATION

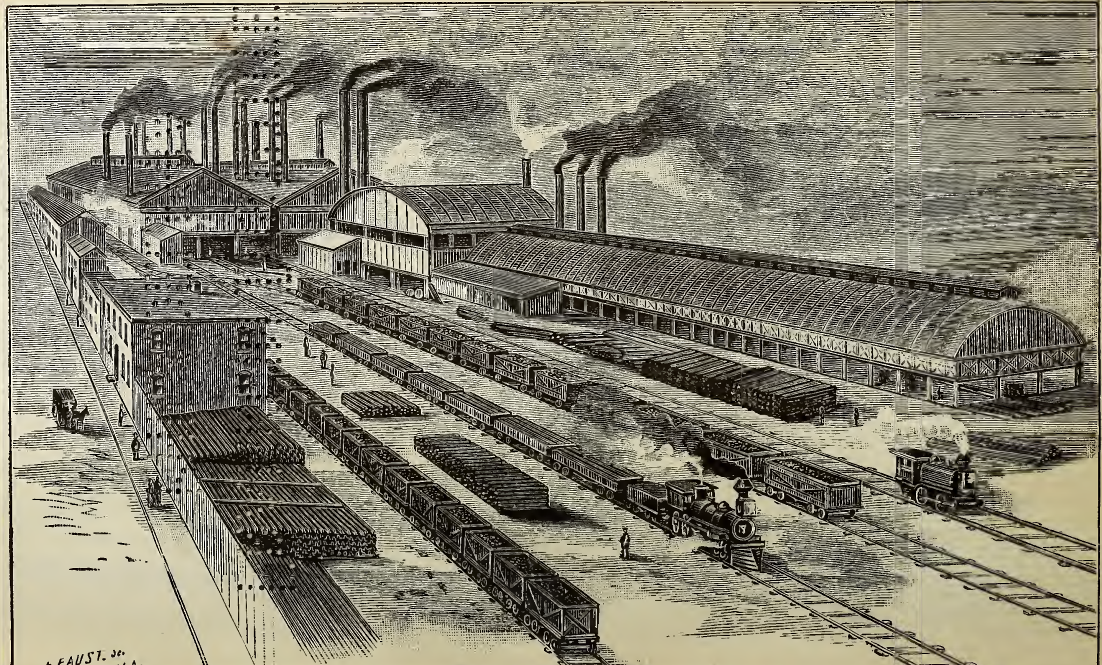
OF

IRON AND STEEL FLOORS, PLATE GIRDERS, ETC., AND OTHER
INFORMATION OF SERVICE TO WORKERS IN IRON.

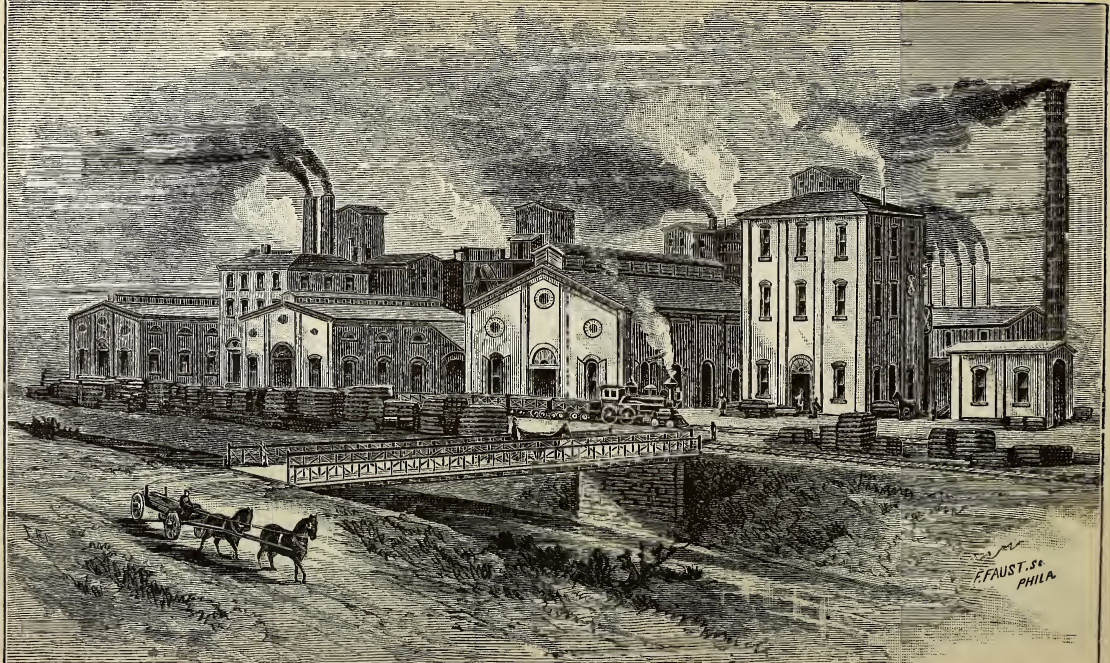
BY
J. C. BLAND,
MEMBER OF AMERICAN SOCIETY OF CIVIL ENGINEERS.

1887.

PRINTED BY
J. B. LIPPINCOTT COMPANY,
PHILADELPHIA.



ROLLING MILL.



FURNACES.

OFFICERS.

C. M. Atkins . . . . President.
William Atkins . . . . Treasurer.
John M. Callen . . . . Secretary.

William Atkins . . . . General Manager.
William Brazier . . . Superintendent of Rolling Mills.
Wm. H. Knowlton . . . . Chief Engineer.
Joseph Summons . . . . Master Mechanic.

CORRESPONDENTS WILL PLEASE ADDRESS

POTTSVILLE IRON AND STEEL CO.,

POTTSVILLE, PENNA.

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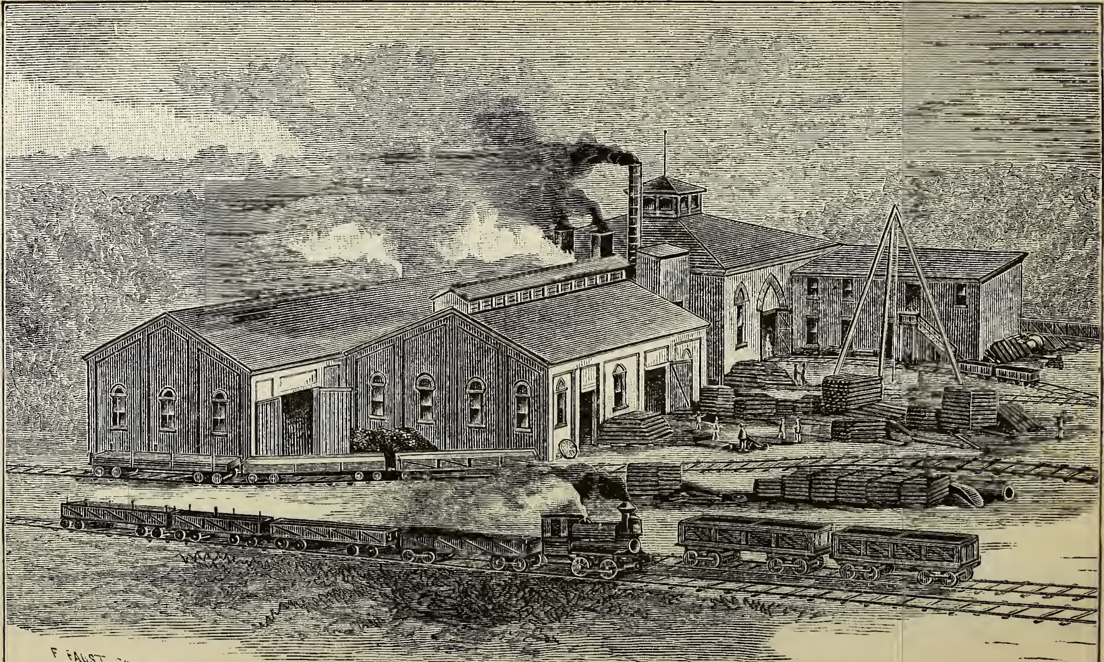
Wm. H. Wallace & Co. . 131 Washington St., New York.
J. F. Bailey . . . . . . 257 S. Fourth St., Philadelphia.
A. G. Tompkins & Co. . . . . 8 Oliver St., Boston, Mass.



MACHINE SHOP AND FOUNDRY.

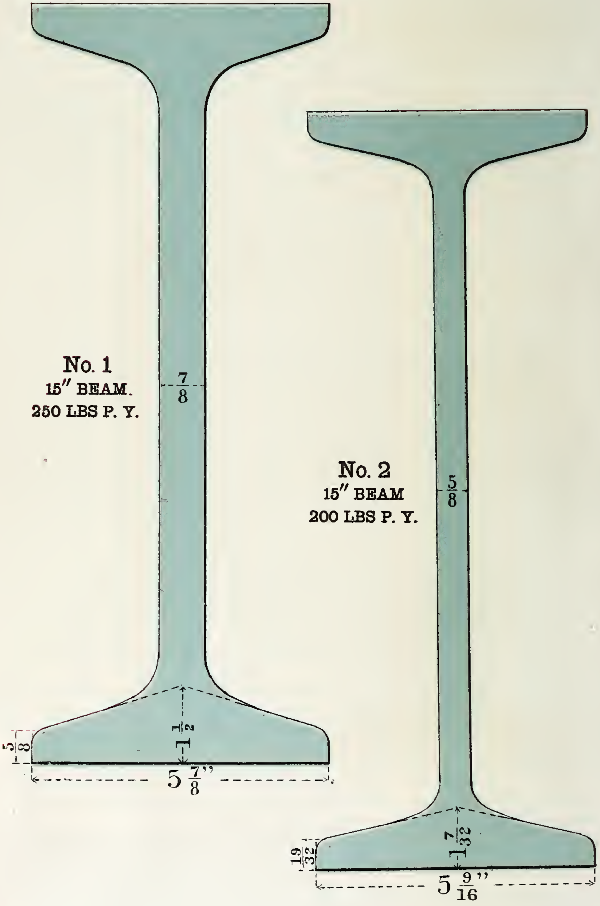
CONTENTS.

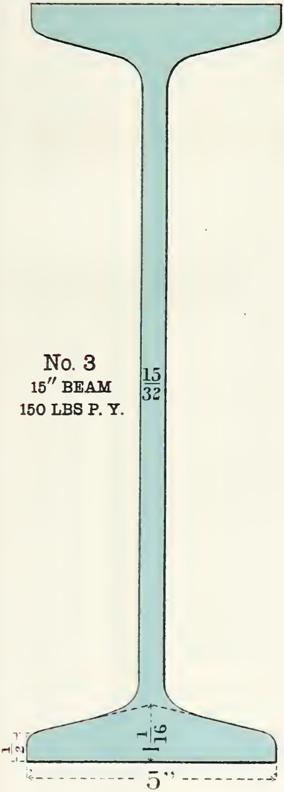
SHAPES OF WROUGHT IRON AND STEEL

MANUFACTURED BY THE

POTTSVILLE IRON AND STEEL COMPANY.

BEAMS.


No. 1
15" BEAM.
250 LBS P. Y.
7/8
1 1/2
5/8
5 7/8"
No. 2
15" BEAM
200 LBS P. Y.
5/8
1 7/32
19/32
5 9/16"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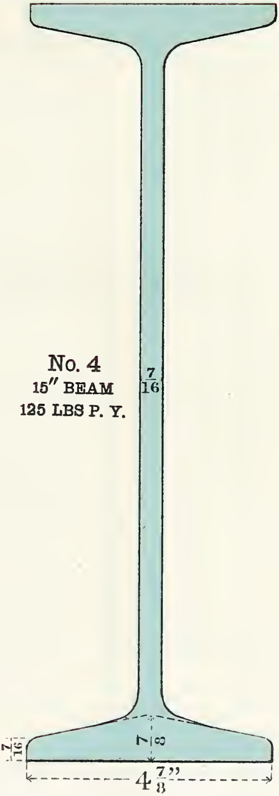
15″ BEAM
150 LBS P. Y.
15/32
1 1/16
1/2
5″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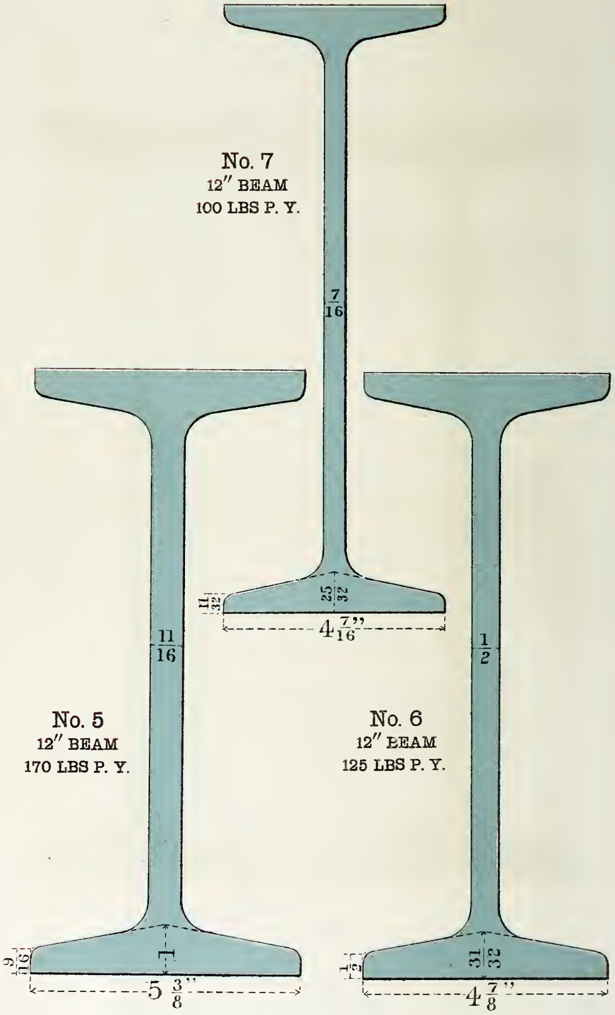
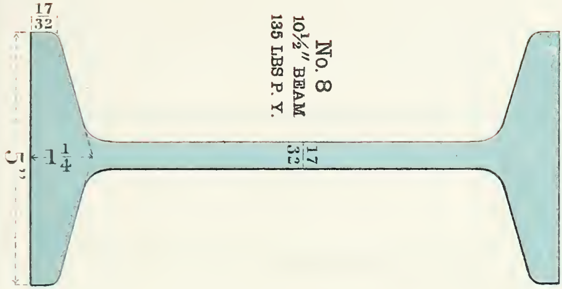
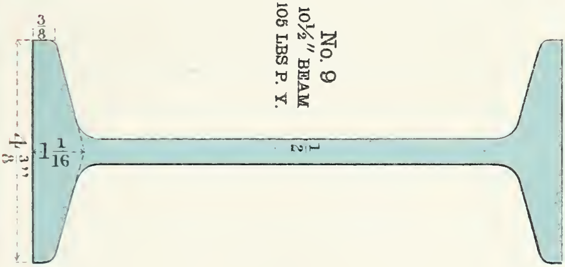
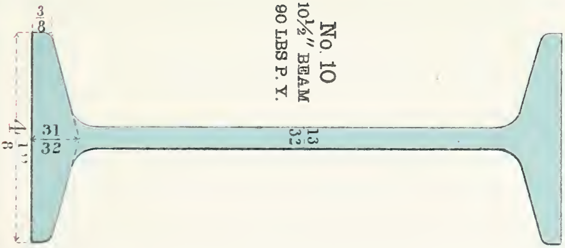
125 LBS P. Y.
7/16
7/8
7/16
4 7/8″


No. 7
12″ BEAM
100 LBS P. Y.
7/16
25/32
11/32
4 7/16″
No. 5
12″ BEAM
170 LBS P. Y.
11/16
1
9/16
5 3/8″
No. 6
12″ BEAM
125 LBS P. Y.
1/2
31/32
1/2
4 7/8″


No. 8
10½″ BEAM
135 LBS P. Y.
17/32
5″
1 1/4
17/32


No. 9
10½″ BEAM
105 LBS P. Y.
3/8
4 3/8″
1 1/16
1/2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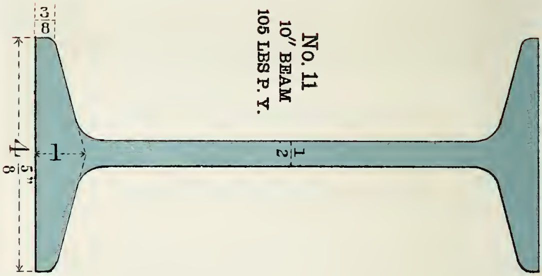
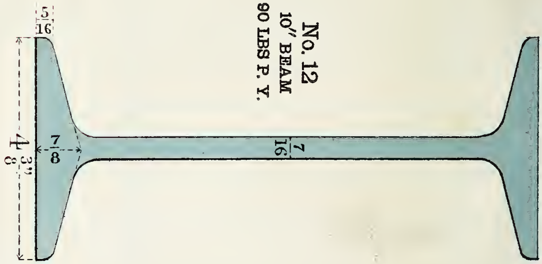
90 LBS P. Y.
3/8
4 1/8″
31/32
13/32


No. 11
10″ BEAM
105 LBS P. Y.
3/8
1
1/2
4 5/8″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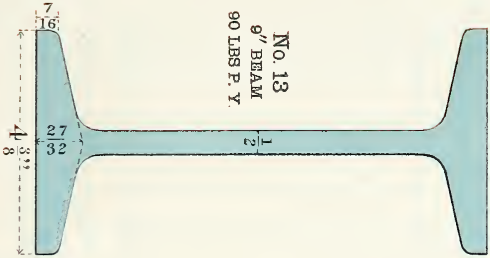
90 LBS P. Y.
5/16
7/8
7/16
4 3/8″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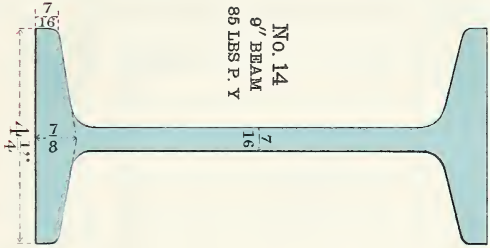
9″ BEAM
90 LBS P. Y.
7/16
4 3/8″
27/32
1/2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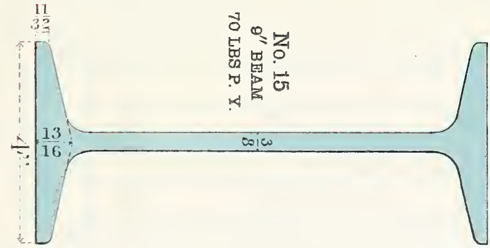
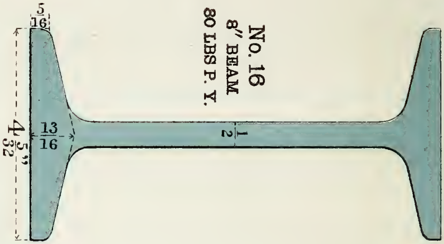
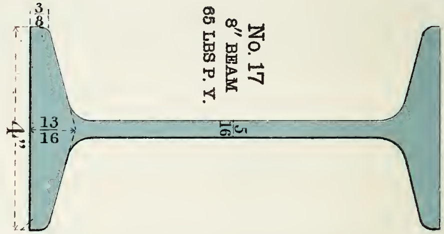
85 LBS P. Y.
7/16
4 1/4″
7/8
7/16

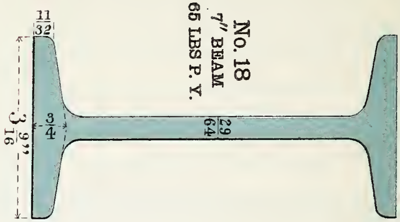

No. 15
9″ BEAM
70 LBS P. Y.
11/32
4″
13/16
3/8



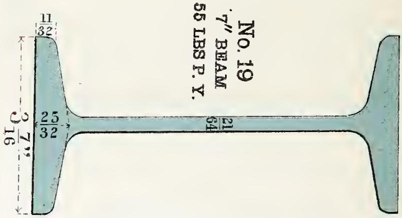
No. 16
8″ BEAM
80 LBS P. Y.



No. 17
8″ BEAM
65 LBS P. Y.

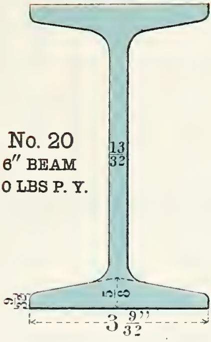


No. 18
7″ BEAM
65 LBS P. Y.

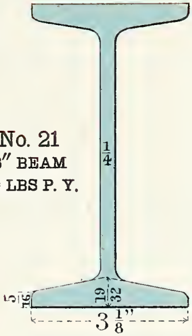


No. 19
7″ BEAM
55 LBS P. Y.

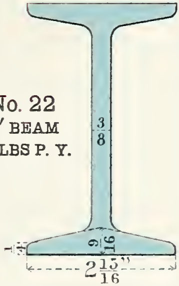
No. 20
6" BEAM
50 LBS P. Y.



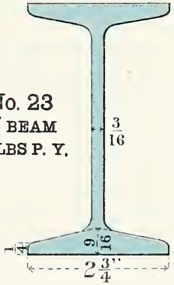
No. 21
6" BEAM
40 LBS P. Y.



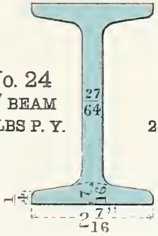
No. 22
5" BEAM
40 LBS P. Y.



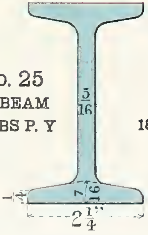
No. 23
5" BEAM
30 LBS P. Y.



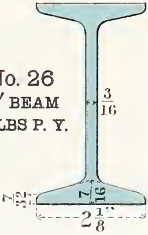
No. 24
4" BEAM
30 LBS P. Y.



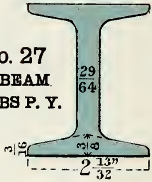
No. 25
4" BEAM
24 LBS P. Y



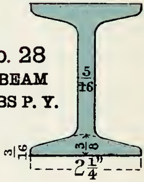
No. 26
4" BEAM
18 LBS P. Y.



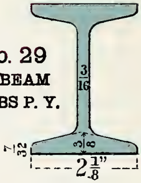
No. 27
3″ BEAM
24 LBS P. Y.



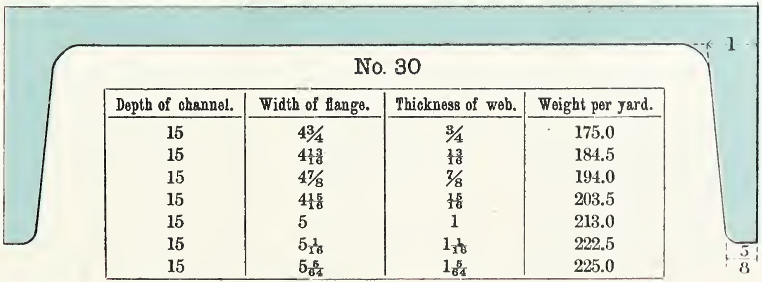
No. 28
3″ BEAM
20 LBS P. Y.



No. 29
3″ BEAM
16 LBS P. Y.

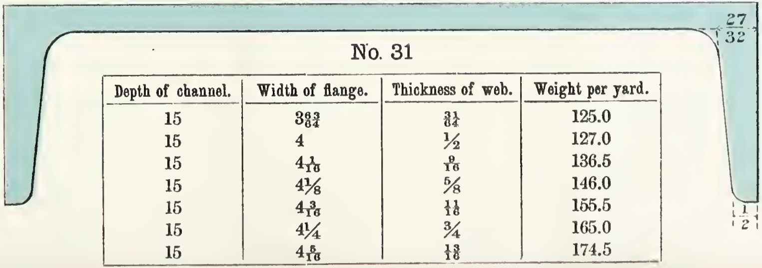


15-INCH CHANNEL.



No. 30

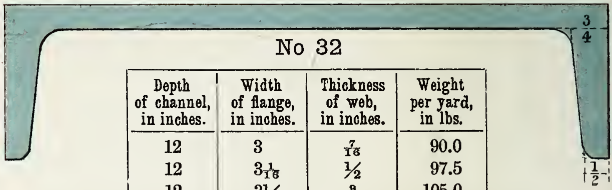
Depth of channel.	Width of flange.	Thickness of web.	Weight per yard.
15	$4\frac{3}{4}$	$\frac{3}{4}$	175.0
15	$4\frac{13}{16}$	$\frac{13}{16}$	184.5
15	$4\frac{7}{8}$	$\frac{7}{8}$	194.0
15	$4\frac{15}{16}$	$\frac{15}{16}$	203.5
15	5	1	213.0
15	$5\frac{1}{16}$	$1\frac{1}{16}$	222.5
15	$5\frac{5}{64}$	$1\frac{5}{64}$	225.0



No. 31

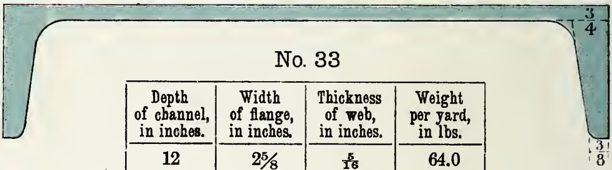
Depth of channel.	Width of flange.	Thickness of web.	Weight per yard.
15	$3\frac{63}{64}$	$\frac{31}{64}$	125.0
15	4	$\frac{1}{2}$	127.0
15	$4\frac{1}{16}$	$\frac{9}{16}$	136.5
15	$4\frac{1}{8}$	$\frac{5}{8}$	146.0
15	$4\frac{3}{16}$	$\frac{11}{16}$	155.5
15	$4\frac{1}{4}$	$\frac{3}{4}$	165.0
15	$4\frac{5}{16}$	$\frac{13}{16}$	174.5

12″



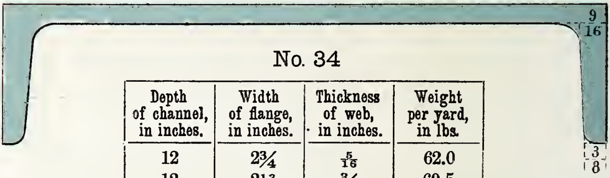
No 32

Depth of channel, in inches.	Width of flange, in inches.	Thickness of web, in inches.	Weight per yard, in lbs.
12	3	$\frac{7}{16}$	90.0
12	$3\frac{1}{16}$	$\frac{1}{2}$	97.5
12	$3\frac{1}{8}$	$\frac{9}{16}$	105.0
12	$3\frac{3}{16}$	$\frac{5}{8}$	112.5
12	$3\frac{1}{4}$	$\frac{11}{16}$	120.0
12	$3\frac{5}{16}$	$\frac{3}{4}$	127.5
12	$3\frac{3}{8}$	$\frac{13}{16}$	135.0
12	$3\frac{7}{16}$	$\frac{7}{8}$	142.5
12	$3\frac{1}{2}$	$\frac{15}{16}$	150.0



No. 33

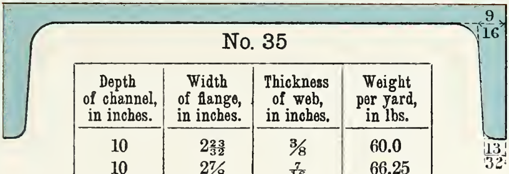
Depth of channel, in inches.	Width of flange, in inches.	Thickness of web, in inches.	Weight per yard, in lbs.
12	$2\frac{5}{8}$	$\frac{5}{16}$	64.0
12	$2\frac{11}{16}$	$\frac{3}{8}$	71.5
12	$2\frac{3}{4}$	$\frac{7}{16}$	79.0
12	$2\frac{13}{16}$	$\frac{1}{2}$	86.5



No. 34

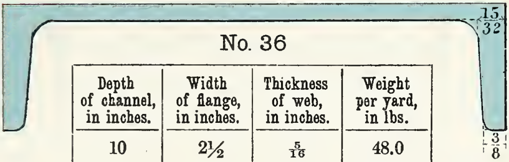
Depth of channel, in inches.	Width of flange, in inches.	Thickness of web, in inches.	Weight per yard, in lbs.
12	$2\frac{3}{4}$	$\frac{5}{16}$	62.0
12	$2\frac{13}{16}$	$\frac{3}{8}$	69.5
12	$2\frac{7}{8}$	$\frac{7}{16}$	77.0
12	$2\frac{15}{16}$	$\frac{1}{2}$	84.5

10″



No. 35

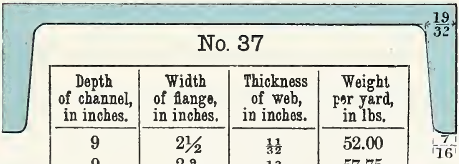
Depth of channel, in inches.	Width of flange, in inches.	Thickness of web, in inches.	Weight per yard, in lbs.
10	$2\frac{23}{32}$	$\frac{3}{8}$	60.0
10	$2\frac{7}{8}$	$\frac{7}{16}$	66.25
10	$2\frac{15}{16}$	$\frac{1}{2}$	72.5
10	3	$\frac{9}{16}$	78.75
10	$3\frac{1}{16}$	$\frac{5}{8}$	85.0
10	$3\frac{1}{8}$	$\frac{11}{16}$	91.25
10	$3\frac{3}{16}$	$\frac{3}{4}$	97.5
10	$3\frac{1}{4}$	$\frac{13}{16}$	103.7
10	$3\frac{5}{16}$	$\frac{7}{8}$	110.0
10	$3\frac{3}{8}$	$\frac{15}{16}$	116.25
10	$3\frac{7}{16}$	1	122.5
10	$3\frac{1}{2}$	$1\frac{1}{16}$	128.75



No. 36

Depth of channel, in inches.	Width of flange, in inches.	Thickness of web, in inches.	Weight per yard, in lbs.
10	$2\frac{1}{2}$	$\frac{5}{16}$	48.0
10	$2\frac{9}{16}$	$\frac{3}{8}$	54.0
10	$2\frac{5}{8}$	$\frac{7}{16}$	62.0

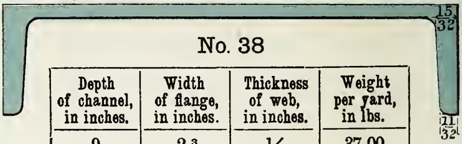
9″



No. 37

Depth of channel, in inches.	Width of flange, in inches.	Thickness of web, in inches.	Weight per yard, in lbs.
9	$2\frac{1}{2}$	$\frac{11}{32}$	52.00
9	$2\frac{9}{16}$	$\frac{13}{32}$	57.75
9	$2\frac{5}{8}$	$\frac{15}{32}$	63.50
9	$2\frac{11}{16}$	$\frac{17}{32}$	69.25
9	$2\frac{3}{4}$	$\frac{19}{32}$	75.00
9	$2\frac{15}{16}$	$\frac{21}{32}$	80.75
9	3	$\frac{23}{32}$	86.50

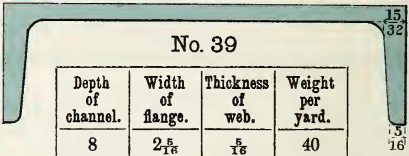
9″



No. 38

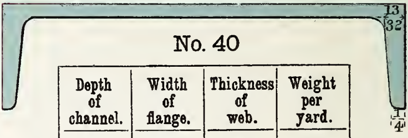
Depth of channel, in inches.	Width of flange, in inches.	Thickness of web, in inches.	Weight per yard, in lbs.
9	$2\frac{3}{16}$	$\frac{1}{4}$	37.00
9	$2\frac{1}{4}$	$\frac{5}{16}$	42.75
9	$2\frac{5}{16}$	$\frac{3}{8}$	48.50
9	$2\frac{3}{8}$	$\frac{7}{16}$	54.25

8″



No. 39

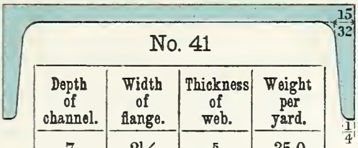
Depth of channel.	Width of flange.	Thickness of web.	Weight per yard.
8	$2\frac{5}{16}$	$\frac{5}{16}$	40
8	$2\frac{3}{8}$	$\frac{3}{8}$	45
8	$2\frac{7}{16}$	$\frac{7}{16}$	50
8	$2\frac{1}{2}$	$\frac{1}{2}$	55
8	$2\frac{9}{16}$	$\frac{9}{16}$	60
8	$2\frac{5}{8}$	$\frac{5}{8}$	65
8	$2\frac{11}{16}$	$\frac{11}{16}$	70



No. 40

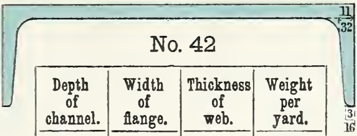
Depth of channel.	Width of flange.	Thickness of web.	Weight per yard.
8	$2\frac{1}{16}$	$\frac{1}{4}$	30
8	$2\frac{1}{8}$	$\frac{5}{16}$	35

7″



No. 41

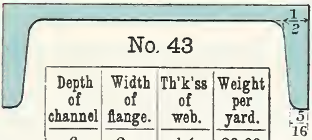
Depth of channel.	Width of flange.	Thickness of web.	Weight per yard.
7	$2\frac{1}{4}$	$\frac{5}{16}$	35.0
7	$2\frac{5}{16}$	$\frac{3}{8}$	39.5
7	$2\frac{3}{8}$	$\frac{7}{16}$	44.0
7	$2\frac{7}{16}$	$\frac{1}{2}$	48.5
7	$2\frac{1}{2}$	$\frac{9}{16}$	53.0
7	$2\frac{9}{16}$	$\frac{5}{8}$	57.5



No. 42

Depth of channel.	Width of flange.	Thickness of web.	Weight per yard.
7	2	$\frac{7}{32}$	25.0
7	$2\frac{1}{16}$	$\frac{9}{32}$	29.5
7	$2\frac{1}{8}$	$\frac{11}{32}$	34.0

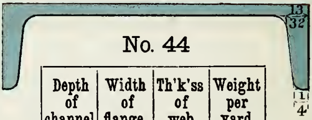
6″



No. 43

Depth of channel	Width of flange.	Th'k'ss of web.	Weight per yard.
6	2	$\frac{1}{4}$	30.00
6	$2\frac{1}{16}$	$\frac{5}{16}$	33.75
6	$2\frac{1}{8}$	$\frac{3}{8}$	37.50
6	$2\frac{3}{16}$	$\frac{7}{16}$	41.25
6	$2\frac{1}{4}$	$\frac{1}{2}$	45.00
6	$2\frac{5}{16}$	$\frac{9}{16}$	48.75
6	$2\frac{3}{8}$	$\frac{5}{8}$	52.50

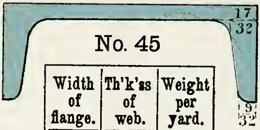
6″



No. 44

Depth of channel	Width of flange.	Th'k'ss of web.	Weight per yard.
6	$1\frac{11}{16}$	$\frac{3}{16}$	22.50
6	$1\frac{3}{4}$	$\frac{1}{4}$	26.25
6	$1\frac{13}{16}$	$\frac{5}{16}$	30.00

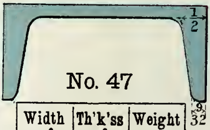
5″



No. 45

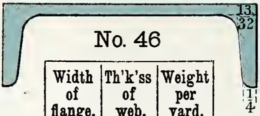
Width of flange.	Th'k'ss of web.	Weight per yard.
$1\frac{7}{8}$	$\frac{1}{4}$	26.00
$1\frac{15}{16}$	$\frac{5}{16}$	29.25
2	$\frac{3}{8}$	32.50
$2\frac{1}{16}$	$\frac{7}{16}$	35.75
$2\frac{1}{8}$	$\frac{1}{2}$	39.00

4″



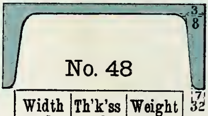
No. 47

Width of flange.	Th'k'ss of web.	Weight per yard.
$1\frac{7}{8}$	$\frac{1}{4}$	24.0
$1\frac{15}{16}$	$\frac{5}{16}$	26.5
2	$\frac{3}{8}$	29.0
$2\frac{1}{16}$	$\frac{7}{16}$	31.5



No. 46

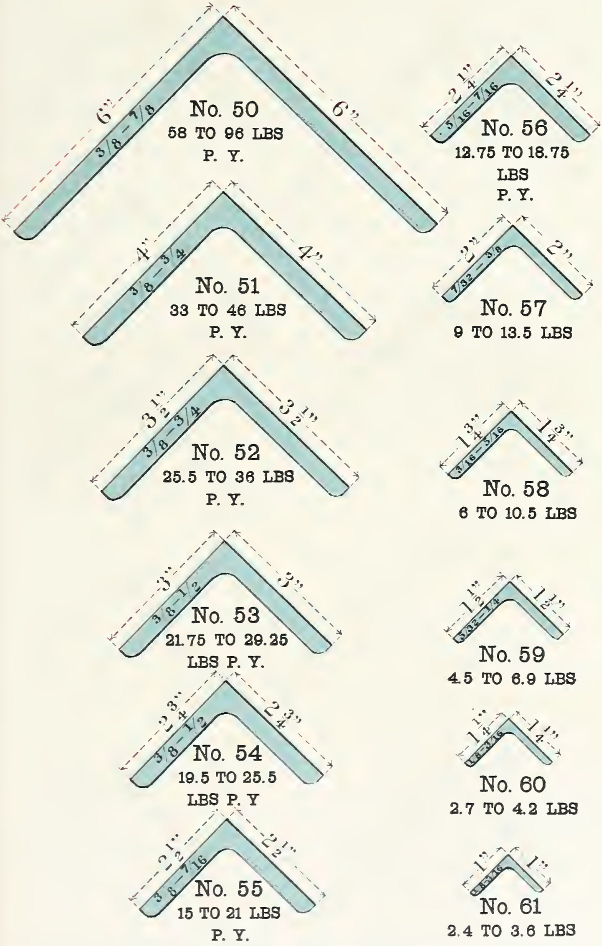
Width of flange.	Th'k'ss of web.	Weight per yard.
$1\frac{5}{8}$	$\frac{3}{16}$	17.00
$1\frac{11}{16}$	$\frac{1}{4}$	20.25
$1\frac{3}{4}$	$\frac{5}{16}$	23.50
$1\frac{13}{16}$	$\frac{3}{8}$	26.75



No. 48

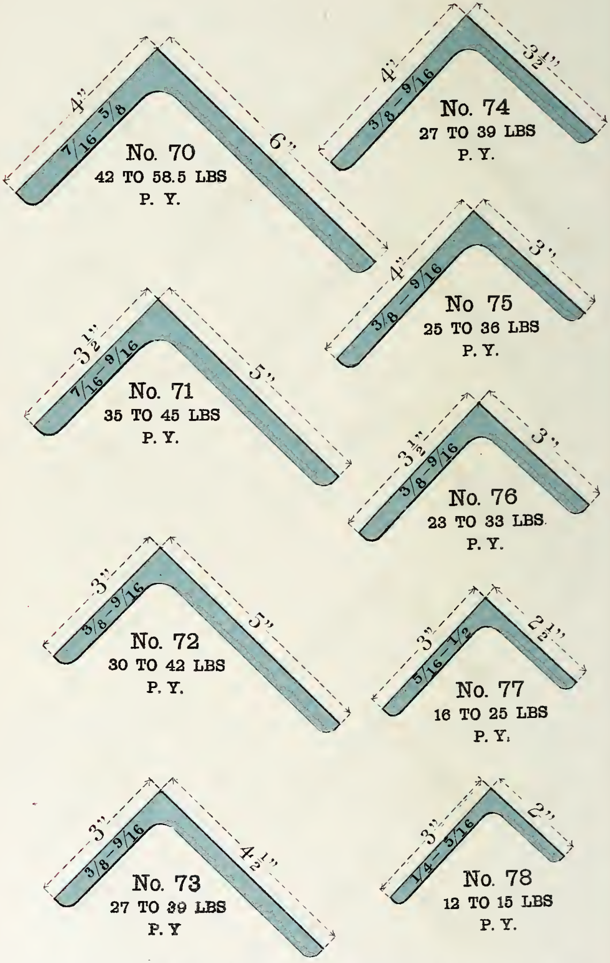
Width of flange.	Th'k'ss of web.	Weight per yard.
$1\frac{5}{8}$	$\frac{3}{16}$	15.0
$1\frac{11}{16}$	$\frac{1}{4}$	17.5
$1\frac{3}{4}$	$\frac{5}{16}$	20.0
$1\frac{13}{16}$	$\frac{3}{8}$	22.5

ANGLES WITH EQUAL LEGS.



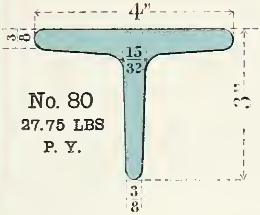
In ordering give either weight or thickness, never both.
Length of leg increases with the weight.

ANGLES WITH UNEQUAL LEGS.

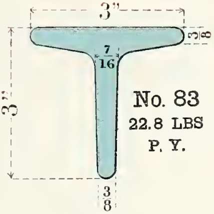


In ordering give either weight or thickness, never both. Length of leg increases with the weight.

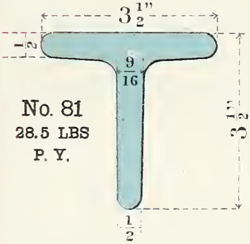
T IRON.



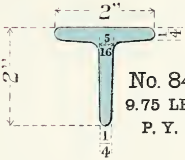
No. 80
27.75 LBS
P. Y.



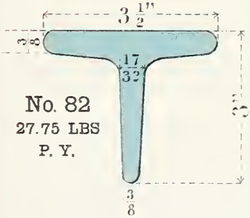
No. 83
22.8 LBS
P. Y.



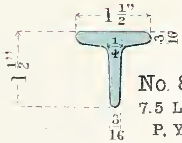
No. 81
28.5 LBS
P. Y.



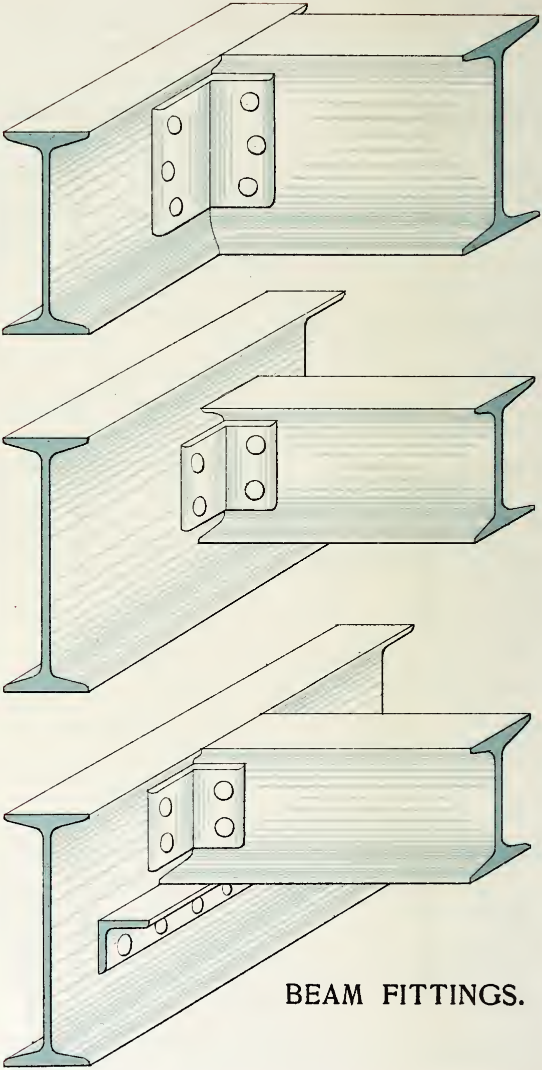
No. 84
9.75 LBS
P. Y.



No. 82
27.75 LBS
P. Y.



No. 85
7.5 LBS
P. Y.

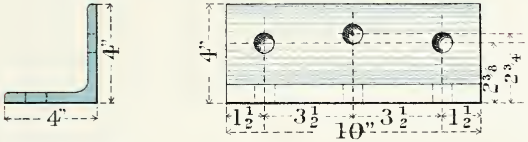


BEAM FITTINGS.

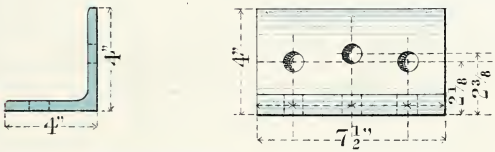
POTTSVILLE IRON AND STEEL CO.'S

STANDARD BRACKETS.

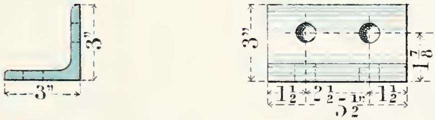
FOR FASTENING BEAMS TO HEADERS.



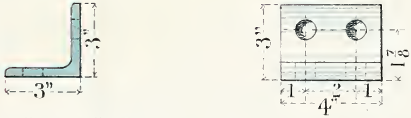
FOR 15″ BEAMS



FOR 12″ AND 10½″ BEAMS



FOR 9″ AND 8″ BEAMS

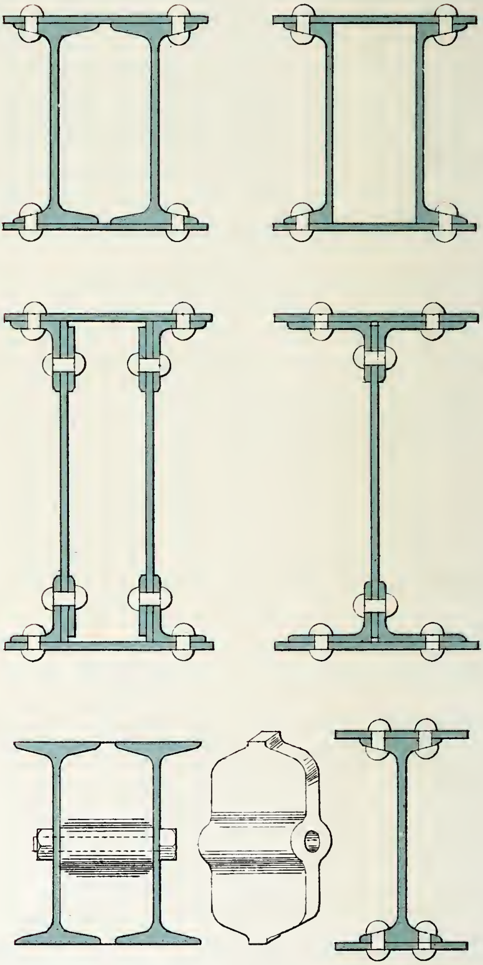


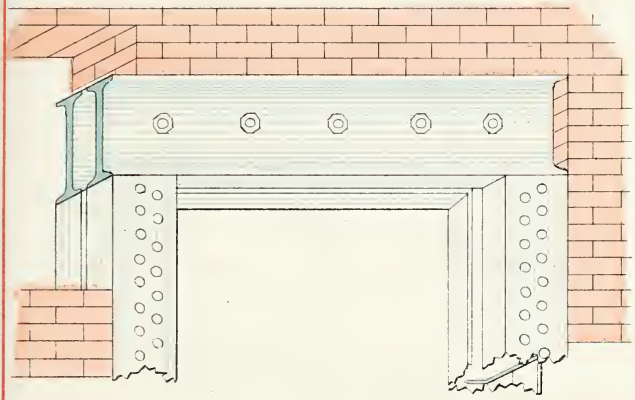
FOR 7″ AND 6″ BEAMS

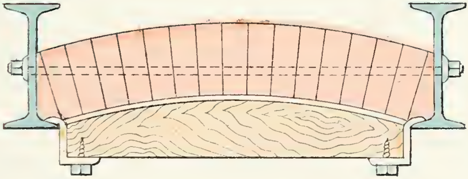
ALL HOLES ARE $\frac{13}{16}$″ DIAMETER FOR ¾″ BOLTS.

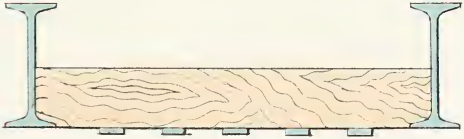
ALL BRACKETS ARE CUT FROM STANDARD ANGLE IRON, EXCEPT WHEN OTHERWISE ORDERED.

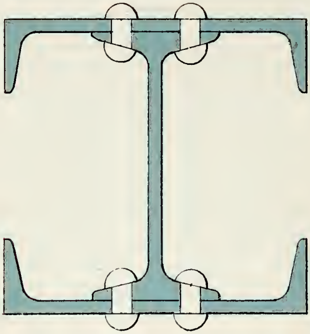
GIRDERS.

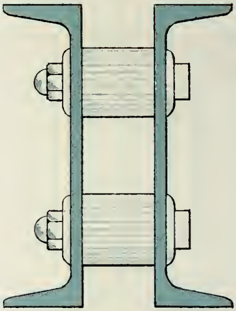


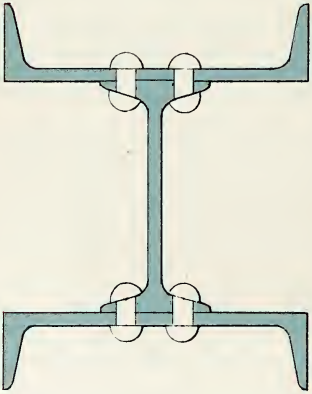


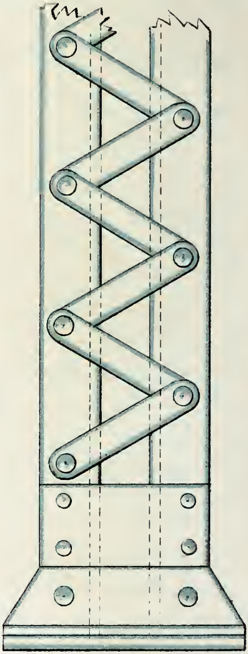






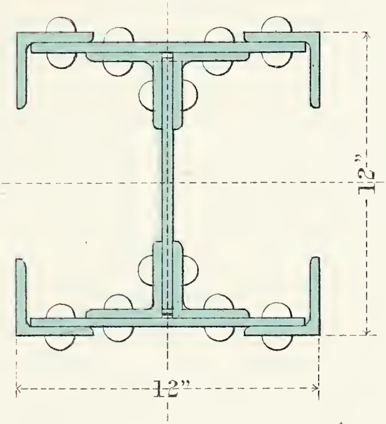


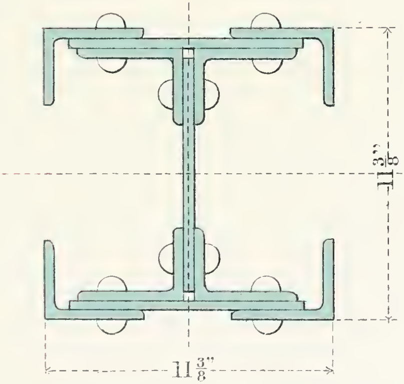




WROUGHT-IRON
COLUMNS.

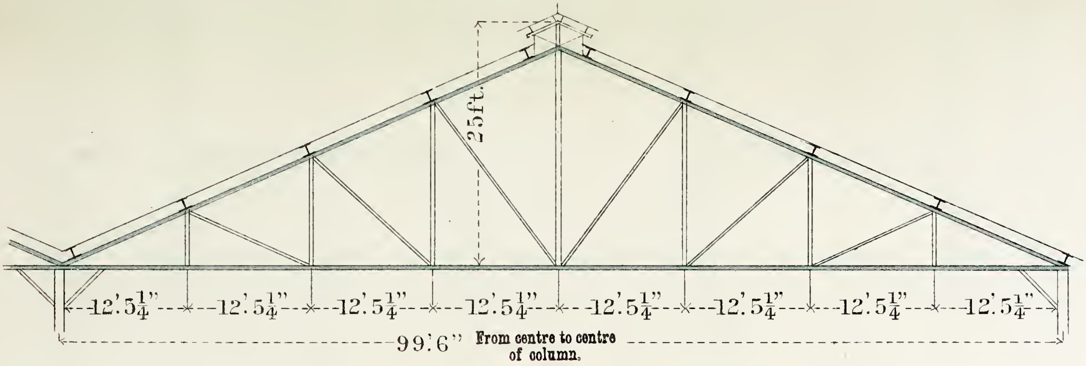
BUILT COLUMNS.





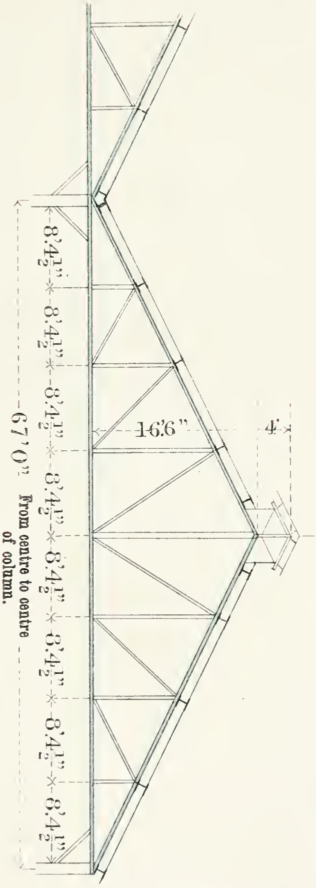
ROOFS FOR POTTSVILLE IRON AND STEEL CO.'S ROLLING MILL, POTTSVILLE

SPAN, 99 FT. 6 IN.　　TRUSSES, 15 FT. APART

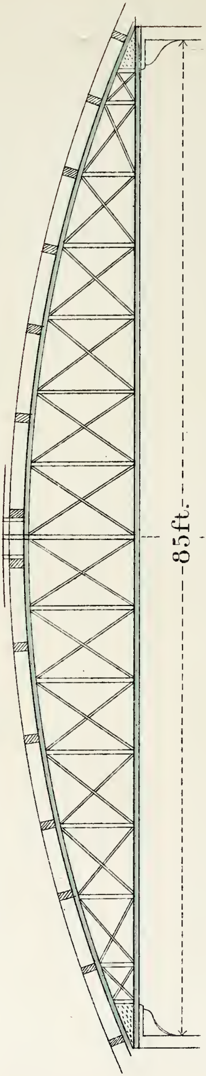


ROOFS FOR POTTSVILLE IRON AND STEEL CO.'S ROLLING MILL, POTTSVILLE.

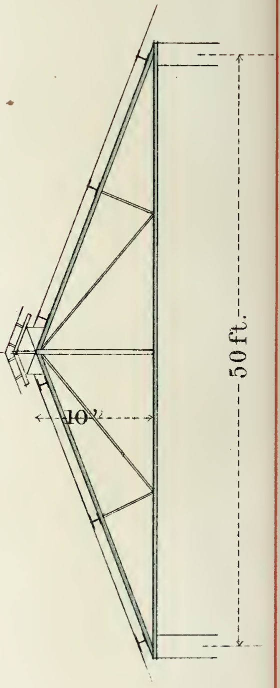
SPAN, 67 FT. 0 IN. TRUSSES, — FT. APART.



ROOFS FOR POTTSVILLE IRON AND STEEL CO.'S ROLLING MILL, POTTSVILLE.



FURNACES, POTTSVILLE.



PRICE CURRENT.

SUBJECT TO CHANGES OF MARKET
WITHOUT NOTICE.

LIST OF REFINED BAR IRON

MADE BY

POTTSVILLE IRON AND STEEL CO.

ORDINARY SIZES.

No Extra.

Round and Square	$\frac{3}{4}$ to 2 in.
Flat Iron	1 to 4 in. × $\frac{3}{8}$ to $1\frac{1}{2}$ in.
Flat Iron	$4\frac{1}{8}$ to 6 in. × $\frac{3}{8}$ to 1 in.

EXTRA SIZES.

Round and Square.	PER LB.	PER TON.
$\frac{5}{8}$ and $\frac{11}{16}$ in.	$\frac{1}{10}$c.	$2 24
$\frac{1}{2}$ and $\frac{9}{16}$ in.	$\frac{2}{10}$c.	4 48
$\frac{7}{16}$ in.	$\frac{4}{10}$c.	8 96
$\frac{3}{8}$ in.	$\frac{5}{10}$c.	11 20
$2\frac{1}{8}$ to $2\frac{7}{8}$ in.	$\frac{1}{10}$c.	2 24
3 to $3\frac{1}{2}$ in.	$\frac{3}{10}$c.	6 72
$3\frac{5}{8}$ to 4 in.	$\frac{5}{10}$c.	11 20
$4\frac{1}{8}$ to $4\frac{1}{2}$ in.	$\frac{6}{10}$c.	13 44
$4\frac{5}{8}$ to 5 in.	$\frac{8}{10}$c.	17 92
$5\frac{1}{4}$ to $5\frac{1}{2}$ in.	1 c.	22 40
$5\frac{3}{4}$ to 6 in.	$1\frac{5}{10}$c.	33 60

EXTRA SIZES.

Flats.	PER LB.	PER TON.
1 to 6 in. × $\frac{1}{4}$ and $\frac{5}{16}$ in.	$\frac{2}{10}$c.	$4 48
1 × $\frac{3}{16}$ in.	$\frac{4}{10}$c.	8 96
4 to 6 in. × $1\frac{1}{8}$ to 2 in.	$\frac{2}{10}$c.	4 48
4 to 6 in. × $2\frac{1}{8}$ to 3 in.	$\frac{4}{10}$c.	8 96
7 × $\frac{3}{8}$ to 1 in.	$\frac{3}{10}$c.	6 72
7 × $1\frac{1}{8}$ to 2 in.	$\frac{4}{10}$c.	8 96
7 × $2\frac{1}{8}$ to 3 in.	$\frac{6}{10}$c.	13 44
8 × $\frac{3}{8}$ to 1 in.	$\frac{4}{10}$c.	8 96
8 × $1\frac{1}{8}$ to $2\frac{3}{4}$ in.	$\frac{6}{10}$c.	13 44
9 × $\frac{3}{8}$ to 1 in.	$\frac{6}{10}$c.	13 44
9 × $1\frac{1}{8}$ to 2 in.	$\frac{8}{10}$c.	17 92
10 × $\frac{3}{8}$ to 1 in.	$\frac{8}{10}$c.	17 92
10 × $1\frac{1}{8}$ to $2\frac{1}{2}$ in.	1 c.	22 40
11 × $\frac{3}{8}$ to 1 in.	$\frac{9}{10}$c.	20 16
11 × $1\frac{1}{8}$ to $2\frac{1}{2}$ in.	$1\frac{1}{10}$c.	24 64
12 × $\frac{3}{8}$ to 1 in.	$\frac{9}{10}$c.	20 16
12 × $1\frac{1}{8}$ to $2\frac{1}{2}$ in.	$1\frac{1}{10}$c.	24 64

6 to 12 in. wide, $\frac{1}{4}$ and $\frac{5}{16}$ in. thick = $\frac{2}{10}$ extra.
For cutting to specified lengths, from $\frac{1}{10}$c. to $\frac{3}{10}$c. per lb.

REMARKS

ON THE

TABLES OF CAPACITY

OF

POTTSVILLE ROLLING MILLS' SHAPES OF IRON AND STEEL.

TABLES OF

BEAMS AND CHANNELS,

Showing the safe load for varying spans, deflexions under the safe load, and proper spacing of shapes for loads varying from 100 to 200 lbs. per square foot.

The first column gives the span in feet.

The second column gives the safe load in nett tons (2000 pounds), uniformly distributed, which the shape will carry for the spans given in the first column, the extreme fibre stress being 6.0 tons per square inch for iron shapes, and 7.8 tons per square inch for steel shapes.

The third column gives the deflexion at centre of span for the safe loads given in second column.

The fourth column gives the weight of the shape for a length equal to the span given in the first column.

The fifth to tenth columns give the maximum distance apart that the shapes can be placed to safely carry loads of

100 to 250 pounds per square foot, the spans being as in the first column.

At the head of each page of the Tables of Capacity are given:

1. The material of which the shape is made.
2. The kind of shape, number, and weight per yard.
3. The depth of shape, width of flange, and thickness of web.
4. The expression for the safe load in nett tons.
5. The maximum shear which the shape can bear without crippling of the web.
6. The span limit,—*i.e.*, the span corresponding to the above maximum shear.

EXTREME FIBRE STRESSES

And reduction of safe loads due to lateral deflexion.

The safe loads given in the following series of tables *include* the weight of the shapes themselves, and assume that *lateral deflexion* does not occur. Should the length of span exceed about *thirty* times the width of flange, the extreme fibre stress should be reduced, or else the shapes should be stayed together. A table is given on page 43, which shows the reduction of fibre stresses in shapes of iron and steel, and likewise gives the proportion of the tabular loads which the shapes will stand, corresponding to the reduced unit stress.

REDUCTION OF

THE EXTREME FIBRE STRESSES

And proportion of the tabular safe loads which must be used when the ratio of span to the flange width of shape exceeds 30.

Ratio of span to flange width of shape.	Corresponding extreme fibre stress for iron shapes.	Corresponding extreme fibre stress for steel shapes.	Proportion of the tabular safe loads which must be used.
30	5.93	7.71	0.99
35	5.71	7.43	0.95
40	5.31	6.90	0.88
45	4.98	6.48	0.83
50	4.67	6.07	0.78
55	4.36	5.67	0.73
60	4.07	5.29	0.68
65	3.79	4.93	0.63
70	3.54	4.60	0.59
75	3.29	4.28	0.55
80	3.07	3.99	0.51
85	2.86	3.72	0.48
90	2.67	3.48	0.45
95	2.50	3.25	0.42
100	2.33	3.03	0.39

The above table is computed from the expression

$$p_c = \frac{f_c}{1 + \frac{1}{5000}\left(\frac{l}{w}\right)^2}$$

where

p_c = reduced fibre stress.
f_c = one-third the modulus of rupture.
$\left.\begin{array}{l} l = \text{length of span} \\ w = \text{flange width} \end{array}\right\}$ Both in same units of dimension.

NOTE.—The exact ratio of span to flange width, for which the fibre stress is that used in the tables, is 28.86.

MAXIMUM SHEAR AND CORRESPONDING SPAN LIMIT.

Besides the capacity of the beam to resist transverse loading, there is also a limit to the load which may be put on a beam, as regards its web resistance. A beam may be amply strong, as concerns its flange area, and yet unable to sustain the load, due to a very thin web.

The maximum shear which a beam can safely bear is determined by the following expressions:

For iron shapes,

$$F_0 = \frac{3.0 \text{ tons}}{1 + \dfrac{\left[\dfrac{h\sqrt{2}}{t}\right]^2}{3000}}$$

For steel shapes,

$$F_0 = \frac{4.0 \text{ tons}}{1 + \dfrac{\left[\dfrac{h\sqrt{2}}{t}\right]^2}{3000}}$$

where h denotes the height of shape in inches, and t denotes the thickness of web in inches.

As for beams under uniformly distributed loads, the end shear F_0 is one-half the total load on the beam, we see that we can load no beam greater than this amount without exceeding the safe shearing stress.

By dividing the coefficient for one foot span by this maximum load, we get the "span limit," and for less spans we cannot use the tabular loads, since they are greater than twice the maximum shear.

The maximum shear and the span limit are given at the head of each Table of Capacity of shapes, and we can see, by inspection of column two in these tables, whether in any case the safe load there given is greater than *twice* the maximum allowable shear. If so, the safe load will be determined by twice the shear value.

If the deflexion of the shape exceeds one-thirtieth ($\frac{1}{30}$) of an inch per foot of span, there is danger of the plaster of the ceiling cracking. This limit has been indicated in the tables by a heavy black line. For spans below this line, shapes should not be used where there is a plaster ceiling, or, if used, the load should be decreased until the corresponding deflexion is less than one-thirtieth ($\frac{1}{30}$) of an inch per foot.

TABLES

OF THE CAPACITY OF

WROUGHT-IRON I BEAMS

UNDER UNIFORMLY DISTRIBUTED
TRANSVERSE LOADS,

THE EXTREME FIBRE STRESS BEING 6.0 TONS PER SQUARE INCH, WHICH IS TWO-SEVENTHS OF

THE MODULUS OF RUPTURE;

AND THE UNSTAYED LENGTH OF FLANGE NOT EXCEEDING THIRTY TIMES ITS WIDTH.

The span, which is *thirty* times the flange width, is denoted by a dotted line on the tables, and for lengths greater than this, the tabular safe load must be reduced by multiplying it by the factors given in table on page 43, or else some method of staying the flanges be employed.

IRON I BEAMS.

15″ I BEAM. SHAPE No. 1. 250 LBS. PER YARD.

Depth, 15″. Width of flange, 5⅞″. Thickness of web, ⅞″.

$$\text{Safe load in nett tons} = \frac{432.00}{\text{Span in feet}}.$$

Maximum shear = 33.06 tons.

Span limit for uniformly distributed load of twice the maximum shear = 6.53′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	43.20	0.09	833						34.55
11	39.27	0.11	917						28.56
12	36.00	0.14	1000				34.28	30.00	24.00
13	33.23	0.16	1083			34.08	29.21	25.56	20.44
14	30.85	0.19	1167		35.25	29.38	25.18	22.03	17.62
15	28.80	0.21	1250	38.40	30.72	25.60	21.94	19.20	15.36
16	27.00	0.24	1333	33.75	27.00	22.50	19.28	16.87	13.50
17	25.41	0.27	1416	29.89	23.91	19.92	17.08	14.94	11.95
18	24.00	0.30	1500	26.66	21.33	17.77	15.23	13.33	10.66
19	22.73	0.33	1583	23.92	19.14	15.95	13.67	11.96	9.57
20	21.60	0.37	1667	21.60	17.28	14.40	12.34	10.75	8.64
21	20.57	0.41	1750	19.59	15.67	13.06	11.19	9.79	7.83
22	19.63	0.45	1833	17.84	14.27	11.89	10.19	8.94	7.13
23	18.78	0.49	1917	16.33	13.06	10.88	9.33	8.16	6.53
24	18.00	0.53	2000	15.00	12.00	10.00	8.57	7.50	6.00
25	17.28	0.58	2083	13.82	11.09	9.21	7.89	6.91	5.54
26	16.61	0.63	2167	12.77	10.23	8.51	7.30	6.38	5.11
27	16.00	0.68	2250	11.85	9.48	7.90	6.77	5.92	4.74
28	15.42	0.73	2333	11.01	8.81	7.34	6.29	5.50	4.40
29	14.89	0.78	2417	10.27	8.21	6.84	5.87	5.13	4.10
30	14.40	0.84	2500	9.60	7.67	6.40	5.48	4.80	3.83
31	13.93	0.90	2583	8.98	7.19	5.98	5.13	4.49	3.59
32	13.50	0.96	2667	8.43	6.75	5.62	4.82	4.21	3.37
33	13.09	1.02	2750	7.93	6.34	5.28	4.53	3.96	3.17

IRON I BEAMS.

15″ I BEAM. SHAPE No. 2. 200 LBS. PER YARD.

Depth, 15″. Width of flange, $5\frac{9}{16}$″. Thickness of web, $\frac{5}{8}$″.

$$\text{Safe load in nett tons} = \frac{370.0}{\text{Span in feet}}.$$

Maximum shear = 20.35 tons.

Span limit for uniformly distributed load of twice the maximum shear = 9.09′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	37.00	0.09	667						29.60
11	33.64	0.11	733					30.58	24.46
12	30.83	0.14	800				29.36	25.69	20.55
13	28.46	0.16	867			29.19	25.02	21.89	17.51
14	26.43	0.19	934		30.21	25.17	21.58	18.88	15.10
15	24.67	0.21	1001	32.89	26.31	21.93	18.79	16.44	13.16
16	23.13	0.24	1067	28.91	23.13	19.27	16.52	14.45	11.56
17	21.76	0.27	1134	25.60	20.48	17.07	14.63	12.80	10.24
18	20.56	0.30	1201	22.84	18.27	15.23	13.05	11.42	9.14
19	19.47	0.33	1267	20.49	16.39	13.66	11.71	10.24	8.20
20	18.50	0.37	1334	18.50	14.80	12.33	10.57	9.25	7.40
21	17.62	0.41	1401	16.78	13.42	11.19	9.59	8.39	6.71
22	16.82	0.45	1467	15.29	12.23	10.19	8.74	7.64	6.12
23	16.09	0.49	1534	13.99	11.19	9.33	7.99	6.99	5.60
24	15.42	0.53	1601	12.85	10.28	8.57	7.34	6.42	5.14
25	14.80	0.58	1668	11.84	9.47	7.89	6.77	5.92	4.74
26	14.23	0.63	1734	10.95	8.76	7.30	6.26	5.47	4.38
27	13.70	0.68	1801	10.15	8.12	6.77	5.80	5.07	4.06
28	13.21	0.73	1868	9.44	7.55	6.29	5.39	4.72	3.78
29	12.76	0.78	1934	8.80	7.04	5.87	5.03	4.40	3.52
30	12.33	0.84	2001	8.22	6.58	5.48	4.70	4.11	3.29
31	11.94	0.90	2068	7.70	6.16	5.13	4.40	3.85	3.08
32	11.56	0.96	2134	7.23	5.78	4.82	4.13	3.61	2.89
33	11.21	1.02	2201	6.79	5.43	4.53	3.88	3.39	2.72

IRON I BEAMS.

15″ I BEAM. SHAPE No. 3. 150 LBS. PER YARD.

Depth, 15″. Width of flange, 5″. Thickness of web, $\frac{15}{32}$″.

$$\text{Safe load in nett tons} = \frac{282.0}{\text{Span in feet}}.$$

Maximum shear = 12.60 tons.

Span limit for uniformly distributed load of twice the maximum shear = 11.19′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	28.20	0.09	500					28.20	22.56
11	25.64	0.11	550				26.64	23.31	18.65
12	23.50	0.14	600			26.11	22.38	19.58	15.67
13	21.69	0.16	650		26.70	22.25	19.07	16.68	13.35
14	20.14	0.19	700	28.77	23.02	19.18	16.44	14.38	11.51
15	18.80	0.21	750	25.07	20.06	16.71	14.33	12.53	10.03
16	17.63	0.24	800	22.04	17.63	14.69	12.59	11.02	8.82
17	16.59	0.27	850	19.52	15.62	13.01	11.15	9.76	7.81
18	15.67	0.30	900	17.41	13.93	11.61	9.95	8.71	6.96
19	14.84	0.33	950	15.62	12.50	10.41	8.92	7.81	6.25
20	14.10	0.37	1000	14.10	11.28	9.40	8.06	7.05	5.64
21	13.43	0.41	1050	12.79	10.23	8.53	7.31	6.39	5.12
22	12.82	0.45	1100	11.65	9.32	7.77	6.66	5.82	4.66
23	12.26	0.49	1150	10.66	8.53	7.11	6.09	5.33	4.26
24	11.75	0.53	1200	9.79	7.83	6.53	5.59	4.89	3.92
25	11.28	0.58	1250	9.02	7.22	6.01	5.15	4.51	3.61
26	10.85	0.63	1300	8.35	6.68	5.57	4.77	4.18	3.34
27	10.44	0.68	1350	7.73	6.18	5.15	4.42	3.86	3.09
28	10.07	0.73	1400	7.19	5.75	4.79	4.11	3.59	2.88
29	9.72	0.78	1450	6.70	5.36	4.47	3.83	3.35	2.68
30	9.40	0.84	1500	6.27	5.02	4.18	3.58	3.13	2.51
31	9.10	0.90	1550	5.87	4.70	3.91	3.35	2.93	2.35
32	8.81	0.96	1600	5.51	4.41	3.67	3.15	2.75	2.20
33	8.55	1.02	1650	5.18	4.14	3.45	2.96	2.59	2.07

IRON I BEAMS.

15″ I BEAM. SHAPE No. 4. 125 LBS. PER YARD.

Depth, 15″. Width of flange, $4\frac{7}{8}$″. Thickness of web, $\frac{7}{16}$″.

$$\text{Safe load in nett tons} = \frac{228.0}{\text{Span in feet}}.$$

Maximum shear = 10.73 tons.

Span limit for uniformly distributed load of twice the maximum shear = 10.62′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	22.80	0.09	417				26.06	22.80	18.24
11	20.73	0.11	458			25.13	21.54	18.84	15.08
12	19.00	0.14	500		25.36	21.11	18.10	15.83	12.67
13	17.54	0.16	542	26.98	21.58	17.99	15.42	13.49	10.79
14	16.29	0.19	583	23.27	18.62	15.51	13.30	11.63	9.31
15	15.20	0.21	625	20.27	16.22	13.51	11.58	10.13	8.11
16	14.25	0.24	667	17.81	14.25	11.87	10.18	8.91	7.12
17	13.41	0.27	709	15.78	12.62	10.52	9.02	7.89	6.31
18	12.67	0.30	750	14.08	11.26	9.39	8.05	7.04	5.63
19	12.00	0.33	792	12.63	10.10	8.42	7.22	6.31	5.05
20	11.40	0.37	834	11.40	9.12	7.60	6.51	5.70	4.56
21	10.86	0.41	875	10.34	8.27	6.89	5.91	5.17	4.14
22	10.36	0.45	917	9.42	7.54	6.28	5.38	4.71	3.77
23	9.91	0.49	959	8.62	6.90	5.75	4.93	4.31	3.45
24	9.50	0.53	1000	7.92	6.34	5.28	4.53	3.96	3.17
25	9.12	0.58	1043	7.30	5.84	4.87	4.17	3.65	2.92
26	8.77	0.63	1084	6.75	5.40	4.50	3.86	3.38	2.70
27	8.44	0.68	1125	6.25	5.00	4.17	3.57	3.13	2.50
28	8.14	0.73	1168	5.81	4.65	3.87	3.32	2.91	2.32
29	7.86	0.78	1209	5.42	4.34	3.61	3.10	2.71	2.17
30	7.60	0.84	1250	5.07	4.06	3.38	2.90	2.53	2.03
31	7.35	0.90	1292	4.74	3.79	3.16	2.71	2.37	1.90
32	7.13	0.96	1334	4.46	3.57	2.97	2.55	2.23	1.78
33	6.91	1.02	1375	4.19	3.35	2.79	2.39	2.09	1.68

IRON I BEAMS.

12″ I BEAM. SHAPE No. 5. 170 LBS. PER YARD.

Depth, 12″. Width of flange, $5\frac{3}{8}$″. Thickness of web, $\frac{11}{16}$″.

$$\text{Safe load in nett tons} = \frac{244.0}{\text{Span in feet}}.$$

Maximum shear = 20.80 tons.

Span limit for uniformly distributed load of twice the maximum shear = 5.86′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	24.40	0.12	567						19.52
11	22.18	0.14	624					20.16	16.13
12	20.33	0.16	680				19.36	16.94	13.55
13	18.77	0.20	737			19.25	16.50	14.43	11.55
14	17.43	0.23	794		19.92	16.60	14.23	12.45	9.96
15	16.27	0.26	850		17.35	14.46	12.39	10.84	8.68
16	15.25	0.30	907	19.06	15.25	12.71	10.89	9.53	7.62
17	14.35	0.34	964	16.88	13.50	11.25	9.65	8.44	6.75
18	13.56	0.38	1021	15.07	12.06	10.05	8.61	7.53	6.03
19	12.84	0.42	1077	13.51	10.81	9.01	7.72	6.75	5.40
20	12.20	0.46	1134	12.20	9.76	8.13	6.97	6.10	4.88
21	11.62	0.51	1190	11.07	8.86	7.38	6.33	5.53	4.43
22	11.09	0.56	1247	10.08	8.06	6.72	5.76	5.04	4.03
23	10.61	0.62	1304	9.23	7.38	6.15	5.27	4.61	3.69
24	10.17	0.67	1361	8.48	6.78	5.65	4.84	4.24	3.39
25	9.76	0.73	1418	7.81	6.25	5.21	4.46	3.91	3.12
26	9.38	0.79	1474	7.22	5.78	4.81	4.13	3.61	2.89
27	9.04	0.84	1530	6.70	5.36	4.47	3.83	3.35	2.68
28	8.71	0.91	1588	6.22	4.98	4.15	3.55	3.11	2.49
29	8.41	0.98	1644	5.80	4.64	3.87	3.31	2.90	2.32
30	8.13	1.05	1700	5.42	4.34	3.61	3.10	2.71	2.17
31	7.87	1.12	1758	5.08	4.06	3.39	2.90	2.54	2.03
32	7.63	1.20	1814	4.77	3.82	3.18	2.73	2.38	1.91
33	7.39	1.27	1871	4.48	3.58	2.99	2.56	2.24	1.79

IRON I BEAMS.

12″ I BEAM. SHAPE No. 6. 125 LBS. PER YARD.

Depth, 12″. Width of flange, 4⅞″. Thickness of web, ½″.

$$\text{Safe load in nett tons} = \frac{185.00}{\text{Span in feet}}.$$

Maximum shear = 13.02 tons.

Span limit for uniformly distributed load of twice the maximum shear = 7.10′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	18.50	0.12	416						14.40
11	16.82	0.14	458					15.35	12.25
12	15.42	0.17	500			17.13	14.69	12.90	10.32
13	14.23	0.20	542			14.65	12.55	10.96	8.77
14	13.21	0.23	583		15.09	12.59	10.79	9.46	7.54
15	12.33	0.26	625		13.15	10.96	9.40	8.25	6.57
16	11.56	0.30	667	14.45	11.61	9.65	8.25	7.22	5.80
17	10.88	0.34	708	12.81	10.27	8.55	7.31	6.40	5.13
18	10.28	0.38	750	11.43	9.15	7.61	6.53	5.71	4.57
19	9.74	0.42	792	10.25	8.21	6.83	5.84	5.12	4.10
20	9.25	0.46	833	9.28	7.40	6.19	5.28	4.64	3.70
21	8.81	0.51	875	8.39	6.70	5.59	4.81	4.19	3.35
22	8.41	0.56	915	7.65	6.10	5.07	4.68	3.82	3.05
23	8.04	0.61	956	7.01	5.59	4.64	3.99	3.50	2.79
24	7.71	0.66	1000	6.45	5.16	4.30	3.67	3.22	2.58
25	7.40	0.72	1042	5.95	4.76	3.95	3.35	2.97	2.38
26	7.12	0.78	1083	5.48	4.38	3.66	3.15	2.74	2.19
27	6.85	0.84	1125	5.07	4.04	3.38	2.88	2.58	2.02
28	6.61	0.91	1167	4.73	3.77	3.14	2.69	2.36	
29	6.38	0.98	1208	4.40	3.52	2.92	2.66	2.20	
30	6.17	1.05	1250	4.12	3.28	2.74	2.35	2.06	
31	5.97	1.12	1292	3.85	3.08	2.53	2.19		
32	5.78	1.19	1333	3.61	2.90	2.41			
33	5.61	1.26	1375	2.69	2.70				

IRON I BEAMS.

12″ I BEAM. SHAPE No. 7. 100 LBS. PER YARD.

Depth, 12″. Width of flange, $4\frac{7}{16}$″. Thickness of web, $\frac{7}{16}$″.

$$\text{Safe load in nett tons} = \frac{144.00}{\text{Span in feet}}.$$

Maximum shear = 10.63 tons.

Span limit for uniformly distributed load of twice the maximum shear = 6.77′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	14.40	0.12	333					14.40	11.52
11	13.09	0.14	367				13.60	11.90	9.52
12	12.00	0.17	400			13.33	11.43	10.00	8.00
13	11.08	0.20	433		13.63	11.36	9.74	8.52	6.81
14	10.28	0.23	467	14.68	11.74	9.79	8.39	7.34	5.87
15	9.60	0.26	500	12.80	10.24	8.53	7.31	6.40	5.12
16	9.00	0.30	533	11.25	9.00	7.50	6.43	5.62	4.50
17	8.47	0.34	567	9.96	7.97	6.64	5.55	4.98	3.98
18	8.00	0.38	600	8.89	7.11	5.93	5.08	4.45	3.55
19	7.58	0.42	633	7.98	6.38	5.32	4.56	3.99	3.19
20	7.20	0.46	667	7.20	5.76	4.80	4.11	3.60	2.88
21	6.86	0.51	700	6.53	5.22	4.35	3.73	3.27	2.61
22	6.55	0.56	733	5.95	4.76	3.97	3.40	2.97	2.38
23	6.26	0.61	767	5.44	4.35	3.63	3.11	2.72	2.17
24	6.00	0.66	800	5.00	4.00	3.33	2.86	2.50	
25	5.76	0.72	833	4.61	3.69	3.07	2.63		
26	5.54	0.78	867	4.26	3.41	2.84			
27	5.33	0.84	900	3.95	3.16	2.63			
28	5.14	0.91	933	3.67	2.94				
29	4.96	0.98	967	3.42	2.74				
30	4.80	1.05	1000	3.20	2.56				
31	4.64	1.12	1033	2.99					
32	4.50	1.19	1067	2.81					
33	4.36	1.26	1100	2.64					

IRON I BEAMS.

10½″ I BEAM. SHAPE No. 8. 135 LBS. PER YARD.

Depth, 10½″. Width of flange, 5″. Thickness of web, $\frac{17}{32}$″.

$$\text{Safe load in nett tons} = \frac{182.00}{\text{Span in feet}}.$$

Maximum shear = 13.27 tons.

Span limit for uniformly distributed load of twice the maximum shear = 6.86′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	18.20	0.14	450						14.50
11	16.55	0.16	495					15.00	12.01
12	15.17	0.19	540				14.40	12.65	10.10
13	14.00	0.23	585			14.32	12.26	10.75	8.61
14	13.00	0.27	630	18.50	14.82	12.34	10.60	9.28	7.40
15	12.13	0.31	675	16.12	12.90	10.75	9.20	8.06	6.45
16	11.37	0.35	720	14.21	11.34	9.45	8.10	7.10	5.67
17	10.71	0.39	765	12.55	9.95	8.35	7.18	6.27	4.97
18	10.11	0.44	810	11.21	8.95	7.47	6.38	5.60	4.47
19	9.58	0.49	855	10.04	8.06	6.67	5.71	5.02	4.03
20	9.10	0.54	900	9.10	7.26	6.05	5.17	4.55	3.63
21	8.67	0.60	945	8.25	6.60	5.50	4.70	4.12	3.30
22	8.27	0.66	990	7.51	6.00	5.00	4.28	3.75	3.00
23	7.91	0.72	1035	6.87	5.50	4.58	3.80	3.43	2.75
24	7.58	0.78	1080	6.30	5.04	4.20	3.60	3.15	2.52
25	7.28	0.85	1125	5.80	4.66	3.86	3.32	2.90	2.33
26	7.00	0.92	1170	5.38	4.28	3.58	3.06	2.69	2.14
27	6.74	0.99	1215	5.00	3.99	3.32	2.90	2.50	
28	6.50	1.07	1260	4.62	3.70	3.09	2.65		
29	6.28	1.14	1305	4.32	3.44	2.90	2.56		
30	6.07	1.22	1350	4.03	3.22	2.69	2.30		
31	5.87	1.30	1395	3.78	2.92	2.52			
32	5.69	1.39	1440	3.55	2.83				
33	5.52	1.48	1485	3.31					

IRON I BEAMS.

10½″ I BEAM. SHAPE No. 9. 105 LBS. PER YARD.

Depth, 10½″. Width of flange, 4⅜″. Thickness of web, ½″.

$$\text{Safe load in nett tons} = \frac{134.00}{\text{Span in feet}}.$$

Maximum shear = 12.13 tons.

Span limit for uniformly distributed load of twice the maximum shear = 5.52′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	13.40	0.14	350					13.40	10.72
11	12.18	0.16	385				12.66	11.07	8.86
12	11.17	0.19	420		14.90	12.41	10.64	9.31	7.45
13	10.31	0.23	455		12.69	10.57	9.06	7.93	6.34
14	9.57	0.27	490	13.67	10.94	9.11	7.81	6.83	5.47
15	8.93	0.31	525	11.91	9.53	7.94	6.81	5.95	4.76
16	8.37	0.35	560	10.46	8.37	6.97	5.98	5.23	4.18
17	7.88	0.39	595	9.27	7.42	6.18	5.30	4.63	3.71
18	7.44	0.44	630	8.27	6.62	5.51	4.73	4.13	3.31
19	7.05	0.49	665	7.42	5.74	4.78	4.10	3.58	2.87
20	6.70	0.54	700	6.70	5.36	4.47	3.83	3.35	2.68
21	6.38	0.60	735	6.08	4.86	4.05	3.47	3.04	2.43
22	6.09	0.66	770	5.54	4.43	3.69	3.17	2.77	
23	5.83	0.72	805	5.07	4.06	3.38	2.90	2.53	
24	5.58	0.78	840	4.65	3.72	3.10	2.66		
25	5.36	0.85	875	4.29	3.43	2.86	2.45		
26	5.15	0.92	910	3.96	3.17	2.64			
27	4.96	0.99	945	3.67	2.94	2 45			
28	4.79	1.07	980	3.42	2.74				
29	4.62	1.14	1015	3.19	2.55				
30	4.47	1.22	1050	2.98					
31	4.32	1.30	1085	2.79					
32	4.19	1.39	1120	2.62					
33	4.06	1.48	1155	2.46					

IRON I BEAMS.

10½″ I BEAM. SHAPE No. 10. 90 LBS. PER YARD.

Depth, 10½″. Width of flange, 4⅛″. Thickness of web, $\frac{13}{32}$″.

$$\text{Safe load in nett tons} = \frac{116.00}{\text{Span in feet}}.$$

Maximum shear = 9.08 tons.

Span limit for uniformly distributed load of twice the maximum shear = 6.39′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	11.60	0.14	300				14.03	11.60	9.28
11	10.55	0.16	330			12.79	10.96	9.59	7.67
12	9.67	0.19	360		12.88	10.74	9.20	8.05	6.44
13	8.92	0.23	390	13.72	10.90	9.08	7.78	6.81	5.49
14	8.29	0.27	420	11.84	9.47	7.89	6.77	5.92	4.74
15	7.73	0.31	450	10.31	8.25	6.87	5.89	5.15	4.12
16	7.25	0.35	480	9.06	7.25	6.04	5.18	4.53	3.62
17	6.82	0.39	510	8.02	6.42	5.35	4.58	4.01	3.21
18	6.44	0.44	540	7.16	5.73	4.77	4.09	3.58	2.86
19	6.11	0.49	570	6.43	5.14	4.29	3.67	3.21	2.57
20	5.80	0.54	600	5.80	4.64	3.87	3.31	2.90	2.32
21	5.52	0.60	630	5.26	4.21	3.51	3.01	2.63	2.10
22	5.27	0.66	660	4.79	3.83	3.19	2.72	2.39	
23	5.04	0.72	690	4.38	3.50	2.92	2.50	2.19	
24	4.83	0.78	720	4.02	3.22	2.68	2.29	2.01	
25	4.64	0.85	750	3.71	2.97	2.47	2.12		
26	4.46	0.92	780	3.43	2.74	2.29			
27	4.30	0.99	810	3.30	2.64	2.12			
28	4.14	1.07	840	2.96	2.37				
29	4.00	1.14	870	2.76	2.21				
30	3.87	1.22	900	2.58	2.06				
31	3.74	1.30	930	2.41					
32	3.62	1.39	960	2.26					
33	3.52	1.48	990	2.13					

IRON I BEAMS.

10″ I BEAM. SHAPE No. 11. 105 LBS. PER YARD.

Depth, 10″. Width of flange, 4⅝″. Thickness of web, ½″.

$$\text{Safe load in nett tons} = \frac{129.00}{\text{Span in feet}}.$$

Maximum shear = 11.90 tons.

Span limit for uniformly distributed load of twice the maximum shear = 5.42′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	12.90	0.15	350					12.90	10.32
11	11.73	0.18	385				12.19	10.67	8.53
12	10.75	0.21	420			11.95	10.24	8.96	7.16
13	9.92	0.24	455		12.21	10.17	8.72	7.63	6.10
14	9.21	0.28	490	13.16	10.53	8.77	7.52	6.58	5.26
15	8.60	0.33	525	11.47	9.18	7.65	6.55	5.74	4.59
16	8.06	0.37	560	10.07	8.06	6.71	5.75	5.03	4.03
17	7.59	0.41	595	8.93	7.14	5.95	5.10	4.47	3.57
18	7.17	0.46	630	7.96	6.37	5.31	4.55	3.98	3.18
19	6.79	0.52	665	6.62	5.30	4.41	3.78	3.31	2.65
20	6.45	0.58	700	6.45	5.16	4.30	3.69	3.22	2.58
21	6.14	0.64	735	5.85	4.68	3.90	3.34	2.92	2.34
22	5.86	0.70	770	5.33	4.26	3.55	3.05	2.66	2.13
23	5.61	0.76	805	4.88	3.90	3.25	2.79	2.44	
24	5.38	0.83	840	4.48	3.58	2.99	2.56	2.24	
25	5.16	0.91	875	4.13	3.30	2.75	2.36	2.07	
26	4.96	0.98	910	3.81	3.05	2.54	2.18		
27	4.78	1.05	945	3.54	2.83	2.36	2.02		
28	4.60	1.13	980	3.29	2.63	2.19			
29	4.44	1.21	1015	3.06	2.45	2.04			
30	4.30	1.29	1050	2.87	2.30				
31	4.16	1.38	1085	2.68	2.14				
32	4.03	1.48	1120	2.52	2.02				
33	3.91	1.58	1155	2.37					

IRON I BEAMS.

10″ I BEAM. SHAPE No. 12. 90 LBS. PER YARD.

Depth, 10″. Width of flange, $4\frac{3}{8}$″. Thickness of web, $\frac{7}{16}$″.

$$\text{Safe load in nett tons} = \frac{111.00}{\text{Span in feet}}.$$

Maximum shear = 9.79 tons.

Span limit for uniformly distributed load of twice the maximum shear = 5.67′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	11.10	0.15	300				12.69	11.10	8.88
11	10.09	0.18	330			12.23	10.49	9.18	7.34
12	9.25	0.21	360		12.33	10.28	8.81	7.71	6.16
13	8.54	0.24	390	13.14	10.51	8.76	7.51	6.57	5.26
14	7.93	0.28	420	11.33	9.06	7.55	6.47	5.67	4.53
15	7.40	0.33	450	9.87	7.89	6.58	5.64	4.93	3.95
16	6.94	0.37	480	8.68	6.94	5.79	4.96	4.34	3.47
17	6.53	0.41	510	7.68	6.14	5.12	4.39	3.84	3.07
18	6.17	0.46	540	6.86	5.48	4.57	3.92	3.43	2.74
19	5.84	0.52	570	6.15	4.92	4.10	3.51	3.08	2.46
20	5.55	0.58	600	5.55	4.44	3.70	3.17	2.78	2.22
21	5.29	0.64	630	5.04	4.03	3.36	2.88	2.52	2.02
22	5.05	0.70	660	4.59	3.67	3.06	2.62	2.29	
23	4.83	0.76	690	4.20	3.36	2.80	2.40	2.10	
24	4.63	0.83	720	3.83	3.06	2.55	2.19		
25	4.44	0.91	750	3.55	2.84	2.37	2.03		
26	4.27	0.98	780	3.28	2.62	2.19			
27	4.11	1.05	810	3.04	2.43	2.03			
28	3.96	1.13	840	2.83	2.26				
29	3.83	1.21	870	2.64	2.11				
30	3.70	1.29	900	2.47					
31	3.58	1.38	930	2.31					
32	3.47	1.48	960	2.17					
33	3.36	1.58	990	2.04					

IRON I BEAMS.

9″ I BEAM. SHAPE No. 13. 90 LBS. PER YARD.

Depth, 9″. Width of flange, 4⅜″. Thickness of web, ½″.

$$\text{Safe load in nett tons} = \frac{98.00}{\text{Span in feet}}.$$

Maximum shear = 11.18 tons.

Span limit for uniformly distributed load of twice the maximum shear = 4.39′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	9.80	0.16	300	19.60	15.68	13.07	11.20	9.80	7.84
11	8.91	0.19	330	16.20	12.96	10.80	9.26	8.10	6.48
12	8.17	0.23	360	13.62	10.90	9.08	7.78	6.81	5.45
13	7.54	0.27	390	11.60	9.28	7.73	6.63	5.80	4.64
14	7.00	0.31	420	10.00	8.00	6.67	5.71	5.00	4.00
15	6.53	0.35	450	8.71	6.97	5.81	4.98	4.35	3.48
16	6.13	0.40	480	7.66	6.13	5.11	4.38	3.83	3.06
17	5.76	0.46	510	6.78	5.42	4.52	3.87	3.39	2.71
18	5.44	0.51	540	6.04	4.83	4.03	3.45	3.02	2.42
19	5.16	0.57	570	5.43	4.34	3.62	3.10	2.71	2.17
20	4.90	0.63	600	4.90	3.92	3.27	2.80	2.45	
21	4.67	0.70	630	4.45	3.56	2.97	2.54	2.22	
22	4.45	0.77	660	4.05	3.24	2.70	2.31	2.02	
23	4.26	0.84	690	3.70	2.96	2.47	2.11		
24	4.08	0.91	720	3.40	2.72	2.27			
25	3.92	0.99	750	3.14	2.51	2.09			
26	3.77	1.07	780	2.90	2.32				
27	3.63	1.16	810	2.69	2.15				
28	3.50	1.24	840	2.50	2.00				
29	3.38	1.33	870	2.33					
30	3.27	1.43	900	2.18					
31	3.16	1.53	930	2.04					
32	3.06	1.63	960						
33	2.97	1.74	990						

IRON I BEAMS.

9″ I BEAM. SHAPE No. 14. 85 LBS. PER YARD.

Depth, 9″. Width of flange, 4¼″. Thickness of web, $\frac{7}{16}$″.

$$\text{Safe load in nett tons} = \frac{96.00}{\text{Span in feet}}.$$

Maximum shear = 9.22 tons.

Span limit for uniformly distributed load of twice the maximum shear = 5.20′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	9.60	0.16	283	19.20	15.36	12.80	10.97	9.60	7.68
11	8.73	0.19	312	15.87	12.70	10.58	9.07	7.93	6.35
12	8.00	0.23	340	13.33	10.66	8.89	7.62	6.66	5.33
13	7.38	0.27	368	12.12	9.70	8.08	6.93	6.06	4.85
14	6.86	0.31	397	9.80	7.84	6.53	5.60	4.90	3.92
15	6.40	0.35	425	8.53	6.82	5.68	4.87	4.26	3.41
16	6.00	0.40	453	7.50	6.00	5.00	4.28	3.75	3.00
17	5.65	0.46	482	6.65	5.32	4.43	3.80	3.32	2.66
18	5.33	0.51	510	5.92	4.73	3.94	3.38	2.96	2.36
19	5.05	0.57	538	5.32	4.25	3.55	3.04	2.66	2.13
20	4.80	0.63	567	4.80	3.84	3.20	2.74	2.40	
21	4.57	0.70	595	4.35	3.48	2.90	2.49	2.17	
22	4.36	0.77	623	3.96	3.17	2.64	2.26		
23	4.17	0.84	652	3.63	2.90	2.42	2.07		
24	4.00	0.91	680	3.33	2.66	2.22			
25	3.84	0.99	708	3.07	2.42				
26	3.69	1.07	737	2.84	2.27				
27	3.56	1.16	765	2.64	2.11				
28	3.43	1.24	793	2.45					
29	3.31	1.33	822	2.28					
30	3.20	1.43	850	2.13					
31	3.10	1.53	878	2.00					
32	3.00	1.63	907						
33	2.91	1.74	935						

IRON I BEAMS.

9″ I BEAM. SHAPE No. 15. 70 LBS. PER YARD.

Depth, 9″. Width of flange, 4″. Thickness of web, ⅜″.

$$\text{Safe load in nett tons} = \frac{74.00}{\text{Span in feet}}.$$

Maximum shear = 7.33 tons.

Span limit for uniformly distributed load of twice the maximum shear = 5.05′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	7.40	0.16	233	14.80	11.84	9.87	8.46	7.40	5.92
11	6.73	0.19	256	12.24	9.79	8.16	6.99	6.12	4.90
12	6.17	0.23	280	10.28	8.22	6.85	5.87	5.14	4.11
13	5.69	0.27	303	8.75	7.00	5.83	5.00	4.37	3.50
14	5.29	0.31	326	7.56	6.05	5.04	4.32	3.78	3.02
15	4.93	0.35	350	6.57	5.26	4.38	3.75	3.28	2.63
16	4.63	0.40	373	5.79	4.63	3.86	3.31	2.89	2.32
17	4.35	0.46	396	5.12	4.10	3.41	2.93	2.56	2.05
18	4.11	0.51	419	4.57	3.66	3.05	2.61	2.28	
19	3.89	0.57	443	4.09	3.27	2.73	2.34	2.04	
20	3.70	0.63	466	3.70	2.96	2.47	2.11		
21	3.52	0.70	489	3.35	2.68	2.23			
22	3.36	0.77	513	3.05	2.44	2.03			
23	3.22	0.84	536	2.80	2.24				
24	3.08	0.91	559	2.57	2.06				
25	2.96	0.99	583	2.37					
26	2.85	1.07	606	2.19					
27	2.74	1.16	629	2.03					
28	2.64	1.24	652						
29	2.55	1.33	676						
30	2.47	1.43	699						
31	2.39	1.53	722						
32	2.31	1.63	746						
33	2.24	1.74	769						

IRON I BEAMS.

8″ I BEAM. SHAPE No. 16. 80 LBS. PER YARD.

Depth, 8″. Width of flange, $4\frac{5}{32}$″. Thickness of web, ½″.

$$\text{Safe load in nett tons} = \frac{77.00}{\text{Span in feet}}.$$

Maximum shear = 10.20 tons.

Span limit for uniformly distributed load of twice the maximum shear = 3.77′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	7.70	0.18	266	15.40	12.32	10.26	8.80	7.70	6.16
11	7.00	0.22	293	12.73	10.18	8.48	7.28	6.36	5.09
12	6.42	0.26	320	10.70	8.56	7.13	6.11	5.35	4.28
13	5.92	0.30	346	9.11	7.29	6.07	5.20	4.55	3.64
14	5.50	0.35	373	7.85	6.28	5.23	4.49	3.92	3.14
15	5.13	0.40	400	6.84	5.47	4.56	3.91	3.42	2.73
16	4.81	0.46	426	6.01	4.80	4.01	3.43	3.00	2.40
17	4.53	0.52	453	5.66	4.53	3.77	3.23	2.83	
18	4.28	0.58	480	4.75	3.80	3.16	2.71	2.37	
19	4.05	0.64	506	4.25	3.40	2.83	2.43		
20	3.85	0.71	532	3.85	3.10	2.56			
21	3.67	0.79	560	3.50	2.80				
22	3.50	0.86	586	3.18	2.54				
23	3.35	0.94	613	2.91					
24	3.21	1.03	640	2.67					
25	3.08	1.12	666	2.46					
26	2.96	1.20	692						
27	2.85	1.30	720						
28	2.75	1.40	746						
29	2.66	1.50	773						
30	2.57	1.60	800						
31	2.48	1.71	826						
32	2.41	1.82	853						
33	2.33	1.93	880						

IRON I BEAMS.

8″ I BEAM. SHAPE No. 17. 65 LBS. PER YARD.

Depth, 8″. Width of flange, 4″. Thickness of web, $\frac{5}{16}$″.

$$\text{Safe load in nett tons} = \frac{68.00}{\text{Span in feet}}.$$

Maximum shear = 5.23 tons.

Span limit for uniformly distributed load of twice the maximum shear = 6.50′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	6.80	0.18	216	13.60	10.88	9.06	7.77	6.80	5.44
11	6.18	0.22	238	11.23	8.98	7.48	6.41	5.62	4.49
12	5.67	0.26	260	9.45	7.56	6.30	5.40	4.72	3.78
13	5.23	0.30	282	8.04	6.43	5.36	4.59	4.02	3.21
14	4.86	0.35	304	6.94	5.55	4.62	3.96	3.47	2.77
15	4.53	0.40	325	6.04	4.83	4.03	3.45	3.02	2.41
16	4.25	0.46	347	5.31	4.25	3.54	3.03	2.66	2.12
17	4.00	0.52	369	4.70	3.76	3.13	2.68	2.35	
18	3.78	0.58	390	4.20	3.36	2.80	2.40		
19	3.58	0.64	412	3.76	3.00	2.51			
20	3.40	0.71	432	3.40	2.72				
21	3.24	0.79	454	3.08	2.46				
22	3.09	0.86	476	2.81					
23	2.96	0.94	498	2.57					
24	2.83	1.03	520						
25	2.72	1.12	542						
26	2.62	1.20	564						
27	2.52	1.30	586						
28	2.43	1.40	608						
29	2.34	1.50	629						
30	2.21	1.60	648						
31	2.19	1.71	672						
32	2.12	1.82	694						
33	2.06	1.93	714						

IRON I BEAMS.

7″ I BEAM. SHAPE No. 18. 65 LBS. PER YARD.

Depth, 7″. Width of flange, $3\frac{9}{16}$″. Thickness of web, $\frac{29}{64}$″.

$$\text{Safe load in nett tons} = \frac{55.00}{\text{Span in feet}}.$$

Maximum shear = 8.18 tons.

Span limit for uniformly distributed load of twice the maximum shear = 3.36′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	5.50	0.20	217	11.00	8.80	7.33	6.29	5.50	4.40
11	5.00	0.25	239	9.09	7.27	6.06	5.19	4.54	3.64
12	4.58	0.29	260	7.63	6.10	5.09	4.36	3.81	3.05
13	4.23	0.35	282	6.51	5.21	4.34	3.72	3.25	2.60
14	3.93	0.40	304	5.61	4.49	3.74	3.21	2.81	2.24
15	3.67	0.46	326	4.89	3.91	3.26	2.79	2.44	
16	3.44	0.52	347	4.30	3.44	2.87	2.46	2.15	
17	3.24	0.59	369	3.81	3.05	2.54	2.18		
18	3.06	0.66	390	3.40	2.72	2.27			
19	2.89	0.74	412	3.04	2.43	2.03			
20	2.75	0.82	434	2.75	2.20				
21	2.62	0.90	456	2.50	2.00				
22	2.50	0.99	477	2.27					
23	2.39	1.08	499	2.08					
24	2.29	1.17	520						
25	2.20	1.27	543						
26	2.12	1.38	564						
27	2.04	1.49	586						
28	1.96	1.60	608						
29	1.90	1.72	629						
30	1.83	1.84	650						
31	1.77	1.96	673						
32	1.72	2.08	694						
33	1.67	2.20	716						

Span limit for tabular safe loads = 9.00′.

IRON I BEAMS.

7″ I BEAM. SHAPE No. 19. 55 LBS. PER YARD.

Depth, 7″. Width of flange, $3\frac{7}{16}$″. Thickness of web, $\frac{21}{64}$″.

Safe load in nett tons $= \dfrac{50.00}{\text{Span in feet}}$.

Maximum shear = 5.31 tons.

Span limit for uniformly distributed load of twice the maximum shear = 4.70′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	5.00	0.20	183	10.00	8.00	6.66	5.71	5.00	4.00
11	4.55	0.25	201	8.27	6.62	5.51	4.72	4.13	3.31
12	4.17	0.29	220	6.95	5.56	4.63	3.97	3.47	2.78
13	3.84	0.35	238	5.90	4.72	3.93	3.37	2.95	2.36
14	3.57	0.40	257	5.10	4.08	3.40	2.91	2.55	2.04
15	3.33	0.46	275	4.44	3.55	2.96	2.53	2.22	
16	3.12	0.52	293	3.90	3.12	2.60			
17	2.94	0.59	312	3.46	2.76	2.30			
18	2.78	0.66	330	3.09	2.47				
19	2.63	0.74	348	2.76	2.21				
20	2.50	0.82	366	2.50					
21	2.38	0.90	385						
22	2.27	0.99	402						
23	2.17	1.08	421						
24	2.08	1.17	440						
25	2.00	1.27	458						
26	1.92	1.38	476						
27	1.85	1.49	495						
28	1.79	1.60	515						
29	1.72	1.72	532						
30	1.67	1.84	550						
31	1.61	1.96	568						
32	1.56	2.08	586						
33	1.52	2.20	605						

Span limit for tabular safe load = 8.70′.

IRON I BEAMS.

6″ I BEAM. SHAPE No. 20. 50 LBS. PER YARD.

Depth, 6″. Width of flange, $3\frac{9}{32}$″. Thickness of web, $\frac{13}{32}$″.

$$\text{Safe load in nett tons} = \frac{36.00}{\text{Span in feet}}.$$

Maximum shear = 6.39 tons.

Span limit for uniformly distributed load of twice the maximum shear = 2.82′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	3.60	0.24	167	7.20	5.76	4.80	4.11	3.60	2.88
11	3.27	0.29	184	5.95	4.76	3.97	3.40	2.97	2.38
12	3.00	0.34	200	5.00	4.00	3.33	2.86	2.50	2.00
13	2.77	0.40	217	4.26	3.41	2.84	2.43	2.13	
14	2.57	0.47	234	3.67	2.94	2.45	2.10		
15	2.40	0.54	250	3.20	2.56	2.13			
16	2.25	0.60	267	2.81	2.25				
17	2.12	0.69	284	2.49					
18	2.00	0.77	300	2.22					
19	1.89	0.86	317						
20	1.80	0.95	334						
21	1.71	1.05	350						
22	1.64	1.15	367						
23	1.57	1.26	384						
24	1.50	1.37	400						
25	1.44	1.49	418						
26	1.38	1.61	434						
27	1.33	1.74	450						
28	1.29	1.87	468						
29	1.24	2.00	484						
30	1.20	2.14	500						
31	1.16	2.27	518						
32	1.13	2.40	534						
33	1.09	2.53	550						

Span limit for tabular safe load = 8.10′.

IRON I BEAMS.

6″ I BEAM. SHAPE No. 21. 40 LBS. PER YARD.

Depth, 6″. Width of flange, $3\frac{1}{8}$″. Thickness of web, $\frac{1}{4}$″.

$$\text{Safe load in nett tons} = \frac{32.00}{\text{Span in feet}}.$$

Maximum shear = 3.30 tons.

Span limit for uniformly distributed load of twice the maximum shear = 4.85′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	3.20	0.24	133	6.40	5.12	4.26	3.65	3.20	2.56
11	2.90	0.29	146	5.26	4.20	3.50	3.01	2.63	2.10
12	2.67	0.34	160	4.45	3.64	2.96	2.54	2.22	
13	2.46	0.40	173	3.78	3.02	2.52	2.16		
14	2.29	0.47	187	3.27	2.61	2.27			
15	2.13	0.54	200	2.84	2.27				
16	2.00	0.60	213	2.50					
17	1.88	0.69	227						
18	1.78	0.77	240						
19	1.68	0.86	253						
20	1.60	0.95	267						
21	1.52	1.05	280						
22	1.45	1.15	293						
23	1.39	1.26	307						
24	1.33	1.37	320						
25	1.28	1.49	333						
26	1.23	1.61	347						
27	1.19	1.74	360						
28	1.14	1.87	373						
29	1.10	2.00	387						
30	1.07	2.14	400						
31	1.03	2.27	413						
32	1.00	2.40	427						
33	0.97	2.53	440						

Span limit for tabular safe load = 7.80′.

IRON I BEAMS.

5″ I BEAM. SHAPE No. 22. 40 LBS. PER YARD.

Depth, 5″. Width of flange, $2\frac{15}{16}$″. Thickness of web, $\frac{3}{8}$″.

$$\text{Safe load in nett tons} = \frac{25.00}{\text{Span in feet}}.$$

Maximum shear = 5.03 tons.

Span limit for uniformly distributed load of twice the maximum shear = 2.48′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
3	8.34	0.02	40	55.60	44.48	37.06	31.77	27.80	22.24
4	6.25	0.04	53	31.25	25.00	20.83	17.83	15.62	12.50
5	5.00	0.07	67	20.00	16.00	13.33	11.43	10.00	8.00
6	4.17	0.10	80	13.90	11.12	9.27	7.94	6.95	5.56
7	3.57	0.14	93	10.20	8.16	6.80	5.83	5.10	4.08
8	3.12	0.18	107	7.80	6.24	5.20	4.46	3.90	3.12
9	2.87	0.23	120	6.38	5.10	4.25	3.65	3.19	2.55
10	2.50	0.28	133	5.00	4.00	3.33	2.85	2.50	2.00
11	2.27	0.34	146	4.13	3.31	2.75	2.36	2.06	
12	2.08	0.41	160	3.47	2.78	2.31			
13	1.92	0.48	173	2.95	2.36				
14	1.79	0.56	187	2.05					
15	1.67	0.64	200						
16	1.56	0.73	213						
17	1.47	0.82	227						
18	1.39	0.92	240						
19	1.32	1.03	253						
20	1.25	1.14	267						
21	1.19	1.26	280						
22	1.14	1.38	293						
23	1.09	1.51	307						
24	1.04	1.65	320						
25	1.00	1.79	333						
26	0.96	1.93	347						

IRON I BEAMS.

5″ I BEAM. SHAPE No. 23. 30 LBS. PER YARD.

Depth, 5″. Width of flange, 2¾″. Thickness of web, $\frac{3}{16}$″.

$$\text{Safe load in nett tons} = \frac{19.20}{\text{Span in feet}}.$$

Maximum shear = 1.90 tons.

Span limit for uniformly distributed load of twice the maximum shear = 5.05′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
3	6.40	0.02	30	42.66	34.12	28.44	24.37	21.33	17.06
4	4.80	0.04	40	24.00	19.20	16.00	13.71	12.00	9.60
5	3.84	0.07	50	15.36	12.28	10.24	8.77	7.68	6.14
6	3.20	0.10	60	10.66	8.52	7.10	6.09	5.33	4.26
7	2.74	0.14	70	7.82	6.25	5.21	4.47	3.91	3.12
8	2.40	0.18	80	6.00	4.80	4.00	3.42	3.00	2.40
9	2.13	0.23	90	4.74	3.79	3.16	2.71	2.37	
10	1.92	0.28	100	3.84	3.08	2.56	2.19		
11	1.75	0.34	110	3.19	2.55	2.12			
12	1.60	0.41	120	2.66	2.12				
13	1.48	0.48	130	2.27					
14	1.37	0.56	140						
15	1.28	0.64	150						
16	1.20	0.73	160						
17	1.13	0.82	170						
18	1.07	0.92	180						
19	1.01	1.03	190						
20	0.96	1.14	200						
21	0.91	1.26	210						
22	0.87	1.38	220						
23	0.83	1.51	230						
24	0.80	1.65	240						
25	0.77	1.79	250						
26	0.74	1.93	260						

IRON I BEAMS.

4″ I BEAM. SHAPE No. 24. 30 LBS. PER YARD.

Depth, 4″. Width of flange, $2\frac{7}{16}$″. Thickness of web, $\frac{27}{64}$″.

$$\text{Safe load in nett tons} = \frac{14.00}{\text{Span in feet}}.$$

Maximum shear = 4.74 tons.

Span limit for uniformly distributed load of twice the maximum shear = 1.45′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
3	4.66	0.03	30	31.16	24.92	20.77	17.80	15.58	12.46
4	3.50	0.06	40	17.50	14.00	11.66	10.00	8.75	7.00
5	2.80	0.09	50	11.20	8.96	7.46	6.40	5.60	4.48
6	2.33	0.13	60	7.77	6.22	5.18	4.44	3.88	3.11
7	2.00	0.17	70	5.71	4.56	3.81	3.26	2.85	2.28
8	1.75	0.23	80	4.37	3.49	2.91	2.49	2.18	
9	1.55	0.29	90	3.22	2.57	2.14			
10	1.40	0.36	100	2.80	2.24				
11	1.27	0.43	110	2.31					
12	1.17	0.51	120						
13	1.08	0.60	130						
14	1.00	0.70	140						
15	0.93	0.81	150						
16	0.87	0.91	160						
17	0.82	1.03	170						
18	0.78	1.16	180						
19	0.74	1.29	190						
20	0.70	1.43	200						
21	0.67	1.58	210						
22	0.64	1.73	220						
23	0.61	1.89	230						
24	0.58	2.06	240						
25	0.56	2.23	250						
26	0.54	2.41	260						

IRON I BEAMS.

4″ I BEAM. SHAPE No. 25. 24 LBS. PER YARD.

Depth, 4″. Width of flange, $2\frac{1}{4}$″. Thickness of web, $\frac{5}{16}$″.

$$\text{Safe load in nett tons} = \frac{11.40}{\text{Span in feet}}.$$

Maximum shear = 3.39 tons.

Span limit for uniformly distributed load of twice the maximum shear = 1.68′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
3	3.80	0.03	24	25.33	20.26	16.89	14.47	12.66	10.13
4	2.85	0.06	32	14.25	11.40	9.50	8.14	7.12	5.70
5	2.28	0.09	40	9.12	7.30	6.08	5.21	4.56	3.65
6	1.90	0.13	48	6.33	5.06	4.22	3.62	3.16	2.53
7	1.63	0.17	56	4.66	3.73	3.11	2.66	2.33	
8	1.43	0.23	64	3.58	2.86	2.39	2.04		
9	1.27	0.29	72	2.82	2.26				
10	1.14	0.36	80	2.28					
11	1.04	0.43	88						
12	0.95	0.51	96						
13	0.88	0.60	104						
14	0.81	0.70	112						
15	0.76	0.81	120						
16	0.71	0.91	128						
17	0.67	1.03	136						
18	0.63	1.16	144						
19	0.60	1.29	152						
20	0.57	1.43	160						
21	0.54	1.58	168						
22	0.52	1.73	176						
23	0.50	1.89	184						
24	0.48	2.06	192						
25	0.46	2.23	200						
26	0.44	2.41	208						

IRON I BEAMS.

4″ I BEAM. SHAPE No. 26. 18 LBS. PER YARD.

Depth, 4″. Width of flange, 2⅛″. Thickness of web, $\frac{3}{16}$″.

$$\text{Safe load in nett tons} = \frac{8.80}{\text{Span in feet}}.$$

Maximum shear = 1.73 tons.

Span limit for uniformly distributed load of twice the maximum shear = 2.54′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
3	2.93	0.03	18	19.53	15.62	13.02	11.16	9.76	7.81
4	2.20	0.06	24	11.00	8.80	7.33	6.29	5.50	4.40
5	1.76	0.09	30	7.04	5.63	4.69	4.02	3.52	2.82
6	1.47	0.13	36	4.90	3.92	3.27	2.80	2.45	
7	1.26	0.17	42	3.60	2.88	2.40	2.06		
8	1.10	0.23	48	2.75	2.20				
9	0.98	0.29	54	2.18					
10	0.88	0.36	60						
11	0.80	0.43	66						
12	0.73	0.51	72						
13	0.68	0.60	78						
14	0.63	0.70	84						
15	0.59	0.81	90						
16	0.55	0.91	96						
17	0.52	1.03	102						
18	0.49	1.16	108						
19	0.46	1.29	114						
20	0.44	1.43	120						
21	0.42	1.58	126						
22	0.40	1.73	132						
23	0.38	1.89	138						
24	0.37	2.06	144						
25	0.35	2.23	150						
26	0.34	2.41	156						

TABLES

OF THE CAPACITY OF

WROUGHT-IRON CHANNELS

UNDER UNIFORMLY DISTRIBUTED
TRANSVERSE LOADS,

THE EXTREME FIBRE STRESS BEING 6.0 TONS PER SQUARE INCH, WHICH IS TWO-SEVENTHS OF

THE MODULUS OF RUPTURE;

AND THE UNSTAYED LENGTH OF FLANGE NOT EXCEEDING THIRTY TIMES ITS WIDTH.

The span, which is *thirty* times the flange width, is denoted by a dotted line on the tables, and for lengths greater than this, the tabular safe load must be reduced by multiplying it by the factors given in table on page 43, or else some method of staying the flanges be employed.

IRON CHANNELS.

15″ CHANNEL. SHAPE No. 30. 225 LBS. PER YARD.

Depth, 15″. Width of flange, $5\frac{5}{64}$″. Thickness of web, $1\frac{5}{64}$″.

$$\text{Safe load in nett tons} = \frac{332.00}{\text{Span in feet}}.$$

Maximum shear = 42.85 tons.

Span limit for uniformly distributed load of twice the maximum shear = 3.88′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	55.33	0.03	450						
8	41.50	0.07	600						
10	33.20	0.11	750				37.94	33.20	26.56
12	27.67	0.15	900		36.90	30.75	26.35	23.06	18.45
14	23.71	0.21	1050	33.87	27.10	22.58	19.35	16.93	13.55
16	20.75	0.27	1200	25.94	20.75	17.29	14.82	12.97	10.38
18	18.44	0.34	1350	20.49	16.39	13.66	11.71	10.24	8.20
20	16.60	0.43	1500	16.60	13.28	11.07	9.49	8.30	6.64
22	15.09	0.52	1650	13.72	10.98	9.15	7.84	6.86	5.49
24	13.83	0.62	1800	11.52	9.22	7.68	6.58	5.76	4.61
26	12.75	0.73	1950	9.81	7.85	6.54	5.61	4.90	3.92
28	11.86	0.84	2100	8.49	6.79	5.66	4.85	4.24	3.40
30	11.07	0.96	2250	7.38	5.90	4.92	4.22	3.69	2.95
32	10.37	1.10	2400	6.48	5.18	4.32	3.70	3.24	2.59
34	9.79	1.25	2550	5.76	4.61	3.84	3.29	2.88	2.30

IRON CHANNELS.

15″ CHANNEL. SHAPE No. 30. 175 LBS. PER YARD.

Depth, 15″. Width of flange, 4¾″. Thickness of web, ¾″.

$$\text{Safe load in nett tons} = \frac{281.00}{\text{Span in feet}}.$$

Maximum shear = 26.98 tons.

Span limit for uniformly distributed load of twice the maximum shear = 5.20′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	46.83	0.03	350						
8	35.13	0.07	467						35.13
10	28.10	0.11	583				32.12	28.10	22.48
12	23.42	0.15	700		31.22	26.02	22.30	19.51	15.61
14	20.07	0.21	817	28.67	22.94	19.11	16.38	14.33	11.47
16	17.56	0.27	933	21.95	17.56	14.63	12.54	10.97	8.78
18	15.61	0.34	1050	17.34	13.87	11.56	9.91	8.67	6.94
20	14.05	0.43	1167	14.05	11.24	9.37	8.03	7.02	5.62
22	12.77	0.52	1283	11.61	9.29	7.74	6.63	5.81	4.64
24	11.71	0.62	1400	9.76	7.81	6.51	5.58	4.88	3.90
26	10.81	0.73	1517	8.32	6.66	5.55	4.75	4.16	3.33
28	10.04	0.84	1633	7.17	5.74	4.78	4.10	3.58	2.87
30	9.37	0.96	1750	6.25	5.00	4.17	3.57	3.12	2.50
32	8.78	1.10	1867	5.49	4.39	3.66	3.14	2.74	2.20
34	8.26	1.25	1983	4.86	3.89	3.24	2.78	2.43	1.94

IRON CHANNELS.

15″ CHANNEL. SHAPE No. 31. 174½ LBS. PER YARD.

Depth, 15″. Width of flange, $4\frac{5}{16}$″. Thickness of web, $\frac{13}{16}$″.

$$\text{Safe load in nett tons} = \frac{265.00}{\text{Span in feet}}.$$

Maximum shear = 29.87 tons.

Span limit for uniformly distributed load of twice the maximum shear = 4.44′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	44.17	0.03	349						
8	33.13	0.07	465					41.41	33.13
10	26.50	0.11	582		42.40	35.33	30.29	26.50	21.20
12	22.08	0.15	698	36.80	29.44	24.53	21.03	18.40	14.72
14	18.93	0.21	814	27.04	21.63	18.03	15.45	13.52	10.81
16	16.56	0.27	931	20.70	16.56	13.80	11.83	10.35	8.28
18	14.72	0.34	1047	16.36	13.09	10.91	9.35	8.18	6.54
20	13.25	0.43	1163	13.25	10.60	8.83	7.57	6.62	5.30
22	12.05	0.52	1280	10.95	8.76	7.30	6.26	5.47	4.38
24	11.04	0.62	1396	9.20	7.36	6.13	5.26	4.60	3.68
26	10.19	0.73	1513	7.84	6.27	5.23	4.48	3.92	3.14
28	9.46	0.84	1629	6.76	5.41	4.51	3.86	3.38	2.70
30	8.83	0.96	1745	5.89	4.71	3.93	3.37	2.94	2.36
32	8.28	1.10	1861	5.18	4.14	3.45	2.96	2.59	2.07
34	7.79	1.25	1978	4.58	3.66	3.05	2.62	2.29	1.83

IRON CHANNELS.

15″ CHANNEL. SHAPE No. 31. 125 LBS. PER YARD.

Depth, 15″. Width of flange, $3\frac{63}{64}$″. Thickness of web, $\frac{31}{64}$″.

$$\text{Safe load in nett tons} = \frac{211.00}{\text{Span in feet}}.$$

Maximum shear = 13.23 tons.

Span limit for uniformly distributed load of twice the maximum shear = 8.00′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs per square foot.	250 lbs. per square foot.
6	35.17	0.03	250						
8	26.38	0.07	333					32.97	26.38
10	21.10	0.11	417		33.76	28.13	24.11	21.10	16.88
12	17.58	0.15	500	29.30	23.44	19.53	16.74	14.65	11.72
14	15.07	0.21	583	21.53	17.22	14.35	12.30	10.76	8.61
16	13.19	0.27	667	16.49	13.19	10.99	9.42	8.24	6.60
18	11.72	0.34	750	13.02	10.42	8.68	7.44	6.51	5.21
20	10.55	0.43	833	10.55	8.44	7.03	6.03	5.27	4.22
22	9.59	0.52	917	8.72	6.98	5.81	4.98	4.36	3.49
24	8.79	0.62	1000	7.33	5.86	4.89	4.19	3.66	2.93
26	8.12	0.73	1083	6.25	5.00	4.17	3.57	3.12	2.50
28	7.54	0.84	1167	5.39	4.31	3.59	3.08	2.69	2.16
30	7.03	0.96	1250	4.69	3.75	3.13	2.68	2.34	
32	6.59	1.10	1333	4.12	3.30	2.75	2.35	2.06	
34	6.21	1.25	1417	3.65	2.92	2.43	2.09		

IRON CHANNELS.

12″ CHANNEL. SHAPE No. 32. 150 LBS. PER YARD.

Depth, 12″. Width of flange, 3½″. Thickness of web, $\frac{15}{16}$″.

$$\text{Safe load in nett tons} = \frac{170.00}{\text{Span in feet}}.$$

Maximum shear = 30.49 tons.

Span limit for uniformly distributed load of twice the maximum shear = 2.80′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	28.33	0.05	300						
8	21.25	0.08	400			35.42	30.36	26.56	21.25
10	17.00	0.13	500	34.00	27.20	22.67	19.43	17.00	13.60
12	14.17	0.19	600	23.62	18.90	15.75	13.50	11.81	9.45
14	12.14	0.26	700	17.34	13.87	11.56	9.91	8.67	6.94
16	10.63	0.34	800	13.29	10.63	8.86	7.59	6.64	5.32
18	9.44	0.43	900	10.49	8.39	6.99	5.99	5.24	4.20
20	8.50	0.54	1000	8.50	6.80	5.67	4.86	4.25	3.40
22	7.73	0.65	1100	7.03	5.62	4.69	4.02	3.51	2.81
24	7.08	0.77	1200	5.90	4.72	3.93	3.37	2.95	2.36
26	6.54	0.90	1300	5.03	4.02	3.35	2.87	2.51	2.01
28	6.07	1.05	1400	4.34	3.47	2.89	2.48	2.17	
30	5.67	1.20	1500	3.78	3.02	2.52	2.16		

IRON CHANNELS.

12″ CHANNEL. SHAPE No. 32. 90 LBS. PER YARD.

Depth, 12″. Width of flange, 3″. Thickness of web, $\frac{7}{16}$″.

$$\text{Safe load in nett tons} = \frac{121.00}{\text{Span in feet}}.$$

Maximum shear = 10.45 tons.

Span limit for uniformly distributed load of twice the maximum shear = 5.79′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	20.17	0.05	180					33.61	26.89
8	15.13	0.08	240		30.26	25.22	21.62	18.91	15.13
10	12.10	0.13	300	24.20	19.36	16.13	13.83	12.10	9.68
12	10.08	0.19	360	16.80	13.44	11.20	9.60	8.40	6.72
14	8.64	0.26	420	12.34	9.87	8.23	7.05	6.17	4.94
16	7.56	0.34	480	9.45	7.56	6.30	5.40	4.72	3.78
18	6.72	0.43	540	7.47	5.98	4.98	4.27	3.73	2.99
20	6.05	0.54	600	6.05	4.84	4.03	3.46	3.02	2.42
22	5.50	0.65	660	5.00	4.00	3.33	2.86	2.50	2.00
24	5.04	0.77	720	4.20	3.36	2.80	2.40	2.10	
26	4.65	0.90	780	3.58	2.86	2.39	2.05		
28	4.32	1.05	840	3.09	2.47	2.06			
30	4.03	1.20	900	2.69	2.15				

IRON CHANNELS.

12″ CHANNEL. SHAPE No. 34. 84½ LBS. PER YARD.

Depth, 12″. Width of flange, $2\frac{15}{16}$″. Thickness of web, ½″.

$$\text{Safe load in nett tons} = \frac{102.00}{\text{Span in feet}}.$$

Maximum shear = 13.00 tons.

Span limit for uniformly distributed load of twice the maximum shear = 3.96′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	17.00	0.05	179				32.38	28.33	22.66
8	12.75	0.08	225		25.50	21.25	18.22	15.92	12.75
10	10.20	0.13	282	20.40	16.32	13.60	11.65	10.20	8.16
12	8.50	0.19	338	14.16	11.33	9.44	8.09	7.08	5.66
14	7.28	0.26	394	10.40	8.32	6.93	5.94	5.20	4.16
16	6.37	0.34	450	7.96	6.37	5.31	4.50	3.98	3.18
18	5.66	0.43	507	6.29	5.03	4.19	3.59	3.14	2.51
20	5.10	0.54	564	5.10	4.08	3.40	2.91	2.55	2.04
22	4.63	0.65	619	4.21	3.36	2.81	2.40	2.11	
24	4.25	0.77	676	3.54	2.83	2.36	2.02		
26	3.92	0.90	732	3.01	2.41	2.01			
28	3.64	1.05	788	2.60	2.09				
30	3.40	1.20	846	2.26					

IRON CHANNELS.

12″ CHANNEL. SHAPE No. 34. 62 LBS. PER YARD.

Depth, 12″. Width of flange, 2¾″. Thickness of web, $\frac{5}{16}$″.

$$\text{Safe load in nett tons} = \frac{84.00}{\text{Span in feet}}.$$

Maximum shear = 5.70 tons.

Span limit for uniformly distributed load of twice the maximum shear = 7.37′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	14.00	0.05	124	46.66	37.33	31.11	26.66	23.33	18.66
8	10.50	0.08	164	26.25	21.00	17.50	15.00	13.12	10.50
10	8.40	0.13	206	16.80	13.44	11.20	9.60	8.40	6.72
12	7.00	0.19	248	11.66	9.33	7.77	6.66	5.83	4.66
14	6.00	0.26	289	8.56	6.85	5.71	4.89	4.28	3.42
16	5.25	0.34	331	6.56	5.25	4.37	3.75	3.28	2.62
18	4.66	0.43	375	5.17	4.14	3.45	2.95	2.59	
20	4.20	0.54	417	4.20	3.36	2.80	2.40		
22	3.82	0.65	454	3.47	2.77	2.31			
24	3.50	0.77	496	2.91	2.33				
26	3.23	0.90	537	2.48					
28	3.00	1.05	578						
30	2.80	1.20	620						

IRON CHANNELS.

10″ CHANNEL. SHAPE No. 35. 128 LBS. PER YARD.

Depth, 10″. Width of flange, $3\frac{1}{2}$″. Thickness of web, $1\frac{1}{16}$″.

$$\text{Safe load in nett tons} = \frac{112.00}{\text{Span in feet}}.$$

Maximum shear = 30.16 tons.

Span limit for uniformly distributed load of twice the maximum shear = 1.86′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs per square foot.	250 lbs. per square foot.
6	18.66	0.04	256					31.10	24.80
8	14.00	0.09	341		28.00	23.33	20.00	17.50	14.00
10	11.20	0.15	426	22.40	17.92	14.93	12.80	11.20	8.96
12	9.33	0.22	512	15.55	12.44	10.36	8.88	7.77	6.22
14	8.00	0.30	597	11.42	9.14	7.62	6.53	5.71	4.57
16	7.00	0.40	682	8.75	7.00	5.83	5.00	4.37	3.50
18	6.22	0.50	768	6.91	5.52	4.61	3.94	3.45	2.76
20	5.60	0.62	852	5.60	4.48	3.73	3.20	2.80	2.24
22	5.09	0.76	938	4.63	3.70	3.08	2.64	2.31	
24	4.66	0.92	1024	3.88	3.11	2.59	2.22		
26	4.31	1.08	1109	3.31	2.59	2.21			
28	4.00	1.24	1194	2.85	2.28				
30	3.73	1.42	1278	2.42					

IRON CHANNELS.

10″ CHANNEL. SHAPE No. 35. 60 LBS. PER YARD.

Depth, 10″. Width of flange, $2\frac{23}{32}$″. Thickness of web, $\frac{3}{8}$″.

$$\text{Safe load in nett tons} = \frac{66.00}{\text{Span in feet}}.$$

Maximum shear = 7.61 tons.

Span limit for uniformly distributed load of twice the maximum shear = 4.34′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	11.00	0.04	120	36.66	29.33	24.44	20.95	18.33	14.66
8	8.25	0.09	160	20.62	16.50	13.75	11.78	10.31	8.25
10	6.60	0.15	200	13.20	10.56	8.80	7.54	6.60	5.28
12	5.50	0.20	240	9.16	7.33	6.11	5.23	4.58	3.66
14	4.71	0.30	280	6.73	5.38	4.48	3.84	3.36	2.69
16	4.12	0.40	320	5.15	4.12	3.43	2.94	2.57	2.06
18	3.66	0.50	360	4.06	3.25	2.72	2.32		
20	3.30	0.62	400	3.30	2.64	2.20			
22	3.00	0.76	440	2.72	2.18				
24	2.75	0.92	480	2.29	1.83				
26	2.53	1.08	520	1.95					
28	2.35	1.24	560						
30	2.20	1.42	600						

IRON CHANNELS.

10″ CHANNEL. SHAPE No. 36. 62 LBS. PER YARD.

Depth, 10″. Width of flange, $2\frac{5}{8}$″. Thickness of web, $\frac{7}{16}$″.

$$\text{Safe load in nett tons} = \frac{64.00}{\text{Span in feet}}.$$

Maximum shear = 9.81 tons.

Span limit for uniformly distributed load of twice the maximum shear = 3.26′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	10.67	0.04	124	35.57	28.46	23.71	20.33	17.78	14.23
8	8.00	0.09	165	20.00	16.00	13.33	11.43	10.00	8.00
10	6.40	0.15	207	12.80	10.24	8.53	7.31	6.40	5.12
12	5.33	0.22	248	8.88	7.10	5.93	5.07	4.44	3.55
14	4.57	0.30	289	6.53	5.22	4.35	3.74	3.27	2.61
16	4.00	0.40	331	5.00	4.00	3.33	2.86	2.50	2.00
18	3.55	0.50	372	3.94	3.15	2.62	2.25	1.97	1.58
20	3.20	0.62	413	3.20	2.56	2.13	1.83	1.60	1.28
22	2.91	0.76	454	2.65	2.12	1.76	1.51	1.32	1.06
24	2.67	0.92	496	2.23	1.78	1.49	1.28	1.12	
26	2.46	1.08	537	1.88	1.50	1.25	1.08		
28	2.29	1.24	579	1.64	1.31	1.09			
30	2.13	1.42	620	1.42	1.14				

IRON CHANNELS.

10″ CHANNEL. SHAPE No. 36. 48 LBS. PER YARD.

Depth, 10″. Width of flange, $2\frac{1}{2}$″. Thickness of web, $\frac{5}{16}$″.

$$\text{Safe load in nett tons} = \frac{52.00}{\text{Span in feet}}.$$

Maximum shear = 5.58 tons.

Span limit for uniformly distributed load of twice the maximum shear = 4.66′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	8.66	0.04	96	28.87	23.09	19.24	16.42	14.43	11.54
8	6.50	0.09	128	16.25	13.00	10.83	9.30	8.12	6.50
10	5.20	0.15	160	10.40	8.32	6.93	5.94	5.20	4.16
12	4.33	0.22	192	7.22	5.77	4.81	4.12	3.61	2.89
14	3.71	0.30	224	5.30	4.24	3.53	3.03	2.65	2.12
16	3.25	0.40	256	4.06	3.25	2.71	2.32		
18	2.88	0.50	288	3.20	2.56				
20	2.60	0.62	320	2.60					
22	2.36	0.76	352						
24	2.17	0.92	384						
26	2.00	1.08	416						
28	1.86	1.24	448						
30	1.73	1.42	480						

IRON CHANNELS.

9″ CHANNEL. SHAPE No. 37. 52 LBS. PER YARD.

Depth, 9″. Width of flange, 2½″. Thickness of web, $\frac{11}{32}$″.

$$\text{Safe load in nett tons} = \frac{53.00}{\text{Span in feet}}.$$

Maximum shear = 6.37 tons.

Span limit for uniformly distributed load of twice the maximum shear = 4.16′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square f.ot.	250 lbs. per square foot.
6	8.83	0.03	104	29.43	23.54	19.62	16.82	14.71	11.77
8	6.63	0.10	139	16.58	13.26	11.05	9.48	8.29	6.63
10	5.30	0.18	173	10.60	8.48	7.07	6.06	5.30	4.24
12	4.41	0.26	208	7.33	5.86	4.89	4.18	3.67	2.93
14	3.78	0.35	243	5.40	4.32	3.60	3.09	2.70	2.16
16	3.31	0.46	277	4.14	3.31	2.76	2.36	2.07	1.65
18	2.95	0.58	312	3.28	2.62	2.19	1.88	1.64	1.31
20	2.65	0.71	347	2.65	2.12	1.77	1.52	1.32	1.06
22	2.41	0.86	381	2.19	1.75	1.46	1.25	1.10	
24	2.20	1.03	416	1.83	1.46	1.22	1.04		
26	2.04	1.20	451	1.57	1.26	1.05			
28	1.90	1.40	485	1.36	1.09				
30	1.77	1.60	520	1.18					

IRON CHANNELS.

9″ CHANNEL. SHAPE No. 38. 37 LBS. PER YARD.

Depth, 9″. Width of flange, $2\frac{3}{16}$″. Thickness of web, ¼″.

$$\text{Safe load in nett tons} = \frac{37.00}{\text{Span in feet}}.$$

Maximum shear = 3.69 tons.

Span limit for uniformly distributed load of twice the maximum shear = 5.01′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	6.17	0.03	74	20.56	16.45	13.71	11.75	10.28	8.23
8	4.63	0.10	87	11.57	9.26	7.71	6.61	5.78	4.63
10	3.70	0.18	123	7.40	5.92	4.93	4.23	3.70	2.96
12	3.17	0.26	148	5.29	4.23	3.53	3.02	2.64	2.13
14	2.64	0.35	173	3.77	3.02	2.51	2.15	1.89	1.51
16	2.31	0.46	197	2.89	2.31	1.93	1.65	1.45	1.16
18	2.06	0.58	222	2.29	1.83	1.53	1.16	1.15	
20	1.85	0.71	247	1.85	1.48	1.23	1.06		
22	1.68	0.86	271	1.53	1.22	1.02			
24	1.54	1.03	296	1.28	1.02				
26	1.42	1.20	321	1.09					
28	1.32	1.40	345	Span limit for tabular safe load = 5.40′.					
30	1.23	1.60	370						

IRON CHANNELS.

8″ CHANNEL. SHAPE No. 39. 40 LBS. PER YARD.

Depth, 8″. Width of flange, $2\frac{5}{16}$″. Thickness of web, $\frac{5}{16}$″.

$$\text{Safe load in nett tons} = \frac{36.00}{\text{Span in feet}}.$$

Maximum shear = 5.25 tons.

Span limit for uniformly distributed load of twice the maximum shear = 3.43′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	6.00	0.05	80	20.00	16.00	13.30	11.42	10.00	8.00
8	4.50	0.11	107	11.25	9.00	7.50	6.42	5.62	4.50
10	3.60	0.20	133	7.20	5.76	4.80	4.11	3.60	2.88
12	3.00	0.30	160	5.00	4.00	3.33	2.85	2.50	2.00
14	2.57	0.40	187	3.70	2.96	2.46	2.11	1.85	1.48
16	2.25	0.50	213	2.80	2.24	1.86	1.60	1.40	1.12
18	2.00	0.66	240	2.22	1.77	1.48	1.26	1.11	
20	1.80	0.80	267	1.80	1.44	1.20	1.02		

IRON CHANNELS.

8″ CHANNEL. SHAPE No. 40. 30 LBS. PER YARD.

Depth, 8″. Width of flange, $2\frac{1}{16}$″. Thickness of web, $\frac{1}{4}$″.

$$\text{Safe load in nett tons} = \frac{26.00}{\text{Span in feet}}.$$

Maximum shear = 3.58 tons.

Span limit for uniformly distributed load of twice the maximum shear = 3.63′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	4.33	0.05	60	14.43	11.54	9.62	8.27	7.21	5.77
8	3.25	0.11	80	8.13	6.50	5.42	4.65	4.07	3.25
10	2.60	0.20	100	5.20	4.16	3.47	2.97	2.60	2.08
12	2.17	0.30	120	3.62	2.90	2.41	2.06	1.81	1.45
14	1.86	0.40	140	2.64	2.11	1.76	1.51	1.32	1.06
16	1.63	0.50	160	2 04	1.63	1.38	1.17	1.02	
18	1.44	0.66	180	1.60	1.28	1.07	Span limit for tabular safe load = 5.10′.		
20	1.30	0.80	200	1.30	1.04				

IRON CHANNELS.

7″ CHANNEL. SHAPE No. 41. 35 LBS. PER YARD.

Depth, 7″. Width of flange, $2\frac{1}{4}$″. Thickness of web, $\frac{5}{16}$″.

$$\text{Safe load in nett tons} = \frac{27.00}{\text{Span in feet}}.$$

Maximum shear = 4.91 tons.

Span limit for uniformly distributed load of twice the maximum shear = 2.75′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	4.50	0.05	70	15.00	12.00	10.00	8.57	7.50	6.00
8	3.37	0.13	93	8.43	6.74	5.62	4.82	4.21	3.37
10	2.70	0.23	117	5.40	4.32	3.60	3.09	2.70	2.16
12	2.25	0.34	140	3.75	3.00	2.50	2.14	1.88	1.50
14	1.93	0.49	163	2.76	2.21	1.84	1.72	1.38	1.11
16	1.68	0.60	187	2.10	1.68	1.40	1.20	1.05	
18	1.50	0.76	210	1.67	1.34	1.11	Span limit for tabular safe load = 5.70′.		
20	1.35	0.94	233	1.35	1.08				

IRON CHANNELS.

7″ CHANNEL. SHAPE No. 42. 25 LBS. PER YARD.

Depth, 7″. Width of flange, 2″. Thickness of web, $\frac{7}{32}$″.

$$\text{Safe load in nett tons} = \frac{20.00}{\text{Span in feet}}.$$

Maximum shear = 2.74 tons.

Span limit for uniformly distributed load of twice the maximum shear = 3.65′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs per square foot.	250 lbs. per square foot.
6	3.33	0.05	50	11.10	8.88	7.40	6.34	5.55	4.44
8	2.75	0.13	67	6.87	5.50	4.58	3.92	3.44	2.75
10	2.00	0.23	83	4.00	3.20	2.67	2.57	2.00	1.60
12	1.67	0.34	100	2.78	2.22	1.85	1.59	1.39	1.11
14	1.43	0.49	117	2.04	1.63	1.36	1.17	1.02	
16	1.25	0.60	133	1.56	1.25	1.04			
18	1.11	0.76	150	1.23	Span limit for tabular safe load = 5.10′.				
20	1.00	0.94	167	1.00					

IRON CHANNELS.

6″ CHANNEL. SHAPE No. 43. 30 LBS. PER YARD.

Depth, 6″. Width of flange, 2″. Thickness of web, ¼″.

Safe load in nett tons $= \frac{22.00}{\text{Span in feet}}$.

Maximum shear = 3.30 tons.

Span limit for uniformly distributed load of twice the maximum shear = 3.30′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	3.33	0.05	60	11.11	8.88	7.40	6.34	5.55	4.44
8	2.75	0.15	80	6.87	5.49	4.58	3.92	3.43	2.74
10	2.20	0.26	100	4.40	3.52	2.93	2.51	2.20	1.76
12	1.83	0.38	120	3.05	2.44	2.03	1.74	1.52	1.22
14	1.57	0.58	140	2.25	1.80	1.50	1.28	1.12	0.90
16	1.38	0.70	160	1.73	1.38	1.15			
18	1.22	0.87	180	1.37	1.09	Span limit for tabular safe load = 5.10′.			
20	1.10	1.08	200	1.10					

IRON CHANNELS.

6″ CHANNEL. SHAPE No. 44. 22½ LBS. PER YARD.

Depth, 6″. Width of flange, $1\frac{11}{16}$″. Thickness of web, $\frac{3}{16}$″.

$$\text{Safe load in nett tons} = \frac{16.00}{\text{Span in feet}}.$$

Maximum shear = 2.00 tons.

Span limit for uniformly distributed load of twice the maximum shear = 4.00′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	2.67	0.05	45	8.90	7.12	5.93	5.08	4.45	3.56
8	2.00	0.15	60	5.00	4.00	3.33	2.85	2.50	2.00
10	1.60	0.26	75	3.20	2.56	2.13	1.83	1.60	1.28
12	1.33	0.38	90	2.22	1.78	1.48	1.26	1.11	
14	1.14	0.58	105	1.63	1.30	1.08			
16	1.00	0.70	120	1.25	1.00				
18	0.89	0.87	135	Span limit for tabular safe load = 4.20′.					
20	0.80	1.08	150						

IRON CHANNELS.

5″ CHANNEL. SHAPE No. 45. 26 LBS. PER YARD.

Depth, 5″. Width of flange, 1⅞″. Thickness of web, ¼″.

$$\text{Safe load in nett tons} = \frac{15.00}{\text{Span in feet}}.$$

Maximum shear = 2.97 tons.

Span limit for uniformly distributed load of twice the maximum shear = 2.53′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	2.50	0.11	52	8.33	6.64	5.55	4.76	4.17	3.32
8	1.88	0.21	69	4.70	3.76	3.13	2.69	2.35	1.88
10	1.50	0.33	87	3.00	2.40	2.00	1.71	1.50	1.20
12	1.25	0.48	104	2.08	1.66	1.38	1.19	1.04	
14	1.07	0.60	121	1.53	1.22	1.02			
16	0.94	0.80	139	1.17	0.94				
18	0.84	1.00	156	Span limit for tabular safe load = 4.80′.					
20	0.75	1.30	173						

IRON CHANNELS.

5″ CHANNEL. SHAPE No. 46. 17 LBS. PER YARD.

Depth, 5″. Width of flange, $1\frac{5}{8}$″. Thickness of web, $\frac{3}{16}$″.

$$\text{Safe load in nett tons} = \frac{10.00}{\text{Span in feet}}.$$

Maximum shear = 1.90 tons.

Span limit for uniformly distributed load of twice the maximum shear = 2.60′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	1.67	0.11	34	5.55	4.44	3.70	3.17	2.78	2.22
8	1.25	0.21	46	3.13	2.50	2.09	1.79	1.57	1.25
10	1.00	0.33	58	2.00	1.60	1.33	1.14	1.00	0.80
12	0.83	0.48	70	1.38	1.10	0.92	0.79	0.69	
14	0.71	0.60	82	1.02	0.82	0.68			
16	0.63	0.80	94	0.79	0.63				
18	0.55	1.00	106	0.61	Span limit for tabular safe load = 4.40′.				
20	0.50	1.30	118	0.50					

TABLES

OF THE CAPACITY OF

STEEL I BEAMS

UNDER UNIFORMLY DISTRIBUTED
TRANSVERSE LOADS,

THE EXTREME FIBRE STRESS BEING 7.8 TONS PER SQUARE INCH, WHICH IS TWO-SEVENTHS OF

THE MODULUS OF RUPTURE;

AND THE UNSTAYED LENGTH OF FLANGE NOT EXCEEDING THIRTY TIMES ITS WIDTH.

The span, which is *thirty* times the flange width, is denoted by a dotted line on the tables, and for lengths greater than this, the tabular safe load must be reduced by multiplying it by the factors given in table on page 43, or else some method of staying the flanges be employed.

STEEL I BEAMS.

15″ I BEAM. SHAPE No. 1. 252½ LBS. PER YARD.

Depth, 15″. Width of flange, 5⅞″. Thickness of web, ⅞″.

Safe load in nett tons = $\frac{563.70}{\text{Span in feet}}$.

Maximum shear = 44.08 tons.

Span limit for uniformly distributed load of twice the maximum shear = 6.39′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	56.36	0.12	842						
11	51.24	0.14	926						
12	46.97	0.18	1010						31.31
13	43.36	0.21	1094					33.35	26.68
14	40.26	0.25	1178				32.86	28.75	23.01
15	37.57	0.27	1263			33.39	28.62	25.04	20.04
16	35.23	0.31	1346		35.23	29.35	25.16	22.02	17.61
17	33.15	0.35	1431	39.00	31.20	26.00	22.28	19.50	15.60
18	31.31	0.39	1515	34.80	27.82	23.20	19.88	17.40	13.91
19	29.66	0.43	1599	31.23	24.97	20.82	17.84	15.61	12.48
20	28.18	0.48	1684	28.18	22.54	18.78	16.10	14.09	11.27
21	26.84	0.53	1767	25.56	20.45	17.04	14.60	12.78	10.23
22	25.62	0.58	1851	23.30	18.62	15.53	13.31	11.65	9.31
23	24.51	0.64	1936	21.31	17.05	14.21	12.18	10.66	8.52
24	23.48	0.69	2020	19.56	15.65	13.04	11.18	9.78	7.82
25	22.54	0.75	2103	18.03	14.42	12.02	10.30	9.02	7.21
26	21.68	0.82	2189	16.68	13.35	11.12	9.53	8.34	6.67
27	20.87	0.88	2261	15.46	12.33	10.31	8.83	7.73	6.17
28	20.13	0.95	2356	14.38	11.50	9.59	8.22	7.19	5.75
29	19.45	1.02	2441	13.41	10.73	8.94	7.66	6.70	5.36
30	18.78	1.08	2525	12.52	10.02	8.35	7.15	6.26	5.01
31	18.18	1.17	2609	11.73	9.38	7.82	6.70	5.86	4.69
32	17.61	1.25	2693	11.01	8.80	7.34	6.29	5.50	4.40
33	17.08	1.33	2777	10.35	8.28	6.90	5.91	5.17	4.14

STEEL I BEAMS.

15″ I BEAM. SHAPE No. 2. 202 LBS. PER YARD.

Depth, 15″. Width of flange, $5\frac{9}{16}$″. Thickness of web, $\frac{5}{8}$″.

$$\text{Safe load in nett tons} = \frac{481.00}{\text{Span in feet}}.$$

Maximum shear = 27.11 tons.

Span limit for uniformly distributed load of twice the maximum shear = 8.87′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	48.10	0.12	673						
11	43.73	0.14	741						31.80
12	40.08	0.18	808					33.36	26.69
13	37.00	0.21	875				32.53	28.46	22.77
14	34.36	0.25	943			32.73	28.05	24.54	19.64
15	32.07	0.27	1010		34.21	28.51	24.43	21.38	17.10
16	30.06	0.31	1077		30.06	25.05	21.47	18.79	15.03
17	28.29	0.35	1145	33.28	26.62	22.19	19.02	16.64	13.31
18	26.72	0.39	1212	29.69	23.75	19.79	16.97	14.84	11.88
19	25.32	0.43	1279	26.64	21.31	17.76	15.22	13.32	10.66
20	24.05	0.48	1347	24.05	19.24	16.03	13.74	12.02	9.62
21	22.90	0.53	1414	21.81	17.45	14.54	12.46	10.90	8.72
22	21.86	0.58	1481	19.87	15.90	13.25	11.35	9.93	7.95
23	20.91	0.64	1549	18.18	14.54	12.12	10.39	9.09	7.27
24	20.04	0.69	1616	16.70	13.36	11.13	9.54	8.35	6.68
25	19.24	0.75	1683	15.39	12.31	10.26	8.79	7.69	6.16
26	18.50	0.82	1751	14.23	11.38	9.49	8.13	7.11	5.69
27	17.81	0.88	1818	13.19	10.55	8.79	7.54	6.55	5.28
28	17.18	0.95	1885	12.27	9.82	8.18	7.01	6.13	4.91
29	16.59	1.02	1953	11.44	9.15	7.63	6.54	5.72	4.58
30	16.03	1.08	2020	10.69	8.55	7.13	6.11	5.34	4.28
31	15.52	1.17	2087	10.01	8.01	6.67	5.72	5.00	4.00
32	15.03	1.25	2155	9.39	7.51	6.26	5.37	4.69	3.76
33	14.58	1.33	2222	8.84	7.07	5.89	5.05	4.42	3.54

STEEL I BEAMS.

15″ I BEAM. SHAPE No. 3. 151½ LBS. PER YARD.

Depth, 15″. Width of flange, 5″. Thickness of web, $\frac{15}{32}$″.

$$\text{Safe load in nett tons} = \frac{366.60}{\text{Span in feet}}.$$

Maximum shear = 16.80 tons.

Span limit for uniformly distributed load of twice the maximum shear = 10.91′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	36.66	0.12	505					36.66	29.33
11	33.33	0.14	556				34.63	30.30	24.24
12	30.55	0.18	606			33.95	29.09	25.46	20.37
13	28.20	0.21	657		34.70	28.92	24.79	21.69	17.35
14	26.19	0.25	707	37.41	29.93	24.94	21.38	18.70	14.97
15	24.44	0.27	757	32.59	26.07	21.73	18.62	16.30	13.04
16	22.91	0.31	808	28.64	22.91	19.09	16.37	14.32	11.46
17	21.56	0.35	859	25.36	20.29	16.91	14.49	12.68	10.15
18	20.37	0.39	909	22.64	18.11	15.09	12.94	11.32	9.06
19	19.29	0.43	959	20.30	16.24	13.53	11.60	10.15	8.12
20	18.33	0.48	1010	18.33	14.66	12.22	10.47	9.17	7.33
21	17.46	0.53	1060	16.62	13.30	11.08	9.50	8.30	6.65
22	16.66	0.58	1111	15.15	12.12	10.10	8.66	7.58	6.06
23	15.94	0.64	1161	13.86	11.09	9.24	7.92	6.93	5.55
24	15.28	0.69	1212	12.74	10.19	8.49	7.28	6.37	5.09
25	14.66	0.75	1263	11.73	9.38	7.82	6.99	5.87	4.69
26	14.10	0.82	1313	10.84	8.67	7.23	6.19	5.42	4.33
27	13.58	0.88	1363	10.06	8.05	6.71	5.75	5.03	4.02
28	13.09	0.95	1414	9.35	7.48	6.23	5.34	4.68	3.74
29	12.64	1.02	1465	8.72	6.98	5.81	4.98	4.36	3.49
30	12.22	1.08	1515	8.15	6.52	5.43	4.66	4.08	3.26
31	11.82	1.17	1565	7.62	6.10	5.08	4.35	3.81	3.05
32	11.46	1.25	1616	7.16	5.73	4.77	4.09	3.58	2.87
33	11.11	1.33	1666	6.74	5.39	4.49	3.85	3.37	2.70

STEEL I BEAMS.

15″ I BEAM. SHAPE No. 4. 126¼ LBS. PER YARD.

Depth, 15″. Width of flange, 4⅞″. Thickness of web, $\frac{7}{16}$″.

$$\text{Safe load in nett tons} = \frac{296.40}{\text{Span in feet}}.$$

Maximum shear = 14.30 tons.

Span limit for uniformly distributed load of twice the maximum shear = 10.36′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	29.64	0.12	421				33.87	29.64	23.71
11	26.94	0.14	463			32.65	27.99	24.49	19.59
12	24.70	0.18	505		32.93	27.44	23.52	20.58	16.46
13	22.80	0.21	547	35.07	28.06	23.38	20.04	17.54	14.03
14	21.17	0.25	589	30.24	24.19	20.16	17.28	15.12	12.10
15	19.76	0.27	631	26.35	21.08	17.57	15.06	13.18	10.54
16	18.53	0.31	673	23.16	18.53	15.41	13.23	11.58	9.27
17	17.44	0.35	715	20.52	16.42	13.68	11.73	10.26	8.21
18	16.47	0.39	757	18.30	14.64	12.20	10.46	9.15	7.32
19	15.60	0.43	800	16.42	13.14	10.95	9.38	8.21	6.57
20	14.82	0.48	842	14.82	11.86	9.88	8.47	7.41	5.93
21	14.11	0.53	884	13.44	10.75	8.96	7.68	6.72	5.38
22	13.47	0.58	926	12.25	9.80	8.17	7.00	6.12	4.90
23	12.89	0.64	968	11.21	8.97	7.47	6.41	5.60	4.48
24	12.35	0.69	1010	10.29	8.23	6.86	5.89	5.15	4.12
25	11.86	0.75	1052	9.49	7.59	6.33	5.42	4.75	3.79
26	11.40	0.82	1094	8.77	7.02	5.85	5.01	4.39	3.51
27	10.98	0.88	1136	8.14	6.51	5.43	4.65	4.07	3.26
28	10.59	0.95	1178	7.56	6.05	5.04	4.32	3.78	3.02
29	10.22	1.02	1220	7.04	5.63	4.69	4.02	3.52	2.81
30	9.88	1.08	1262	6.59	5.27	4.39	3.77	3.30	2.63
31	9.56	1.17	1305	6.17	4.94	4.12	3.52	3.08	2.47
32	9.26	1.25	1347	5.79	4.63	3.86	3.31	2.80	2.31
33	8.98	1.33	1389	5.44	4.35	3.63	3.11	2.72	2.18

STEEL I BEAMS.

12″ I BEAM. SHAPE No. 5. 171¾ LBS. PER YARD.

Depth, 12″. Width of flange, 5⅜″. Thickness of web, $\frac{11}{16}$″.

$$\text{Safe load in nett tons} = \frac{317.20}{\text{Span in feet}}.$$

Maximum shear = 27.72 tons.

Span limit for uniformly distributed load of twice the maximum shear = 5.72′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	31.72	0.15	573				36.25	31.72	25.38
11	28.84	0.18	630			34.96	29.97	26.22	20.98
12	26.43	0.22	687		35.24	29.37	25.17	22.02	17.62
13	24.40	0.26	744	35.23	28.18	23.49	20.13	17.62	14.09
14	22.66	0.30	802	32.37	25.90	21.58	18.50	16.18	12.95
15	21.15	0.34	859	28.20	22.56	18.80	16.11	14.10	11.28
16	19.82	0.39	916	24.78	19.82	16.52	14.16	12.39	9.91
17	18.66	0.44	973	21.95	17.56	14.63	12.54	10.97	8.78
18	17.62	0.49	1030	19.58	15.66	13.05	11.19	9.79	7.83
19	16.71	0.55	1088	17.59	14.07	11.73	10.05	8.80	7.04
20	15.86	0.59	1145	15.86	12.69	10.57	9.06	7.93	6.34
21	15.10	0.66	1202	14.38	11.50	9.59	8.22	7.19	5.75
22	14.42	0.73	1260	13.11	10.49	8.74	7.49	6.55	5.24
23	13.79	0.79	1317	11.99	9.59	7.99	6.85	6.00	4.80
24	13.22	0.86	1374	11.02	8.82	7.35	6.30	5.51	4.41
25	12.69	0.94	1431	10.15	8.12	6.77	5.80	5.07	4.06
26	12.20	1.01	1489	9.38	7.50	6.25	5.36	4.69	3.75
27	11.75	1.09	1546	8.70	6.96	5.80	4.97	4.35	3.48
28	11.33	1.18	1603	8.09	6.47	5.39	4.62	4.05	3.24
29	10.94	1.27	1660	7.54	6.03	5.03	4.31	3.77	3.02
30	10.57	1.36	1718	7.05	5.64	4.70	4.03	3.52	2.82
31	10.23	1.46	1775	6.60	5.28	4.40	3.77	3.30	2.64
32	9.92	1.55	1832	6.20	4.96	4.13	3.54	3.10	2.48
33	9.61	1.64	1889	5.82	4.66	3.88	3.33	2.91	2.33

STEEL I BEAMS.

12″ I BEAM. SHAPE No. 6. 126¼ LBS. PER YARD.

Depth, 12″. Width of flange, 4⅞″. Thickness of web, ½″.

$$\text{Safe load in nett tons} = \frac{240.90}{\text{Span in feet}}.$$

Maximum shear = 17.34 tons.

Span limit for uniformly distributed load of twice the maximum shear = 6.94′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	24.10	0.15	421			32.13	27.54	24.10	19.28
11	21.90	0.18	463		31.86	26.54	22.75	19.91	15.93
12	20.08	0.22	505	33.47	26.77	22.31	19.12	16.73	13.38
13	18.53	0.26	548	28.51	22.81	19.01	16.29	14.25	11.40
14	17.21	0.30	590	24.58	19.66	16.39	14.05	12.29	9.83
15	16.06	0.34	632	21.41	17.13	14.27	12.24	10.71	8.57
16	15.06	0.39	674	18.83	15.06	12.55	10.76	9.41	7.53
17	14.17	0.44	716	16.67	13.34	11.12	9.53	8.34	6.67
18	13.39	0.49	758	14.88	11.90	9.92	8.50	7.44	5.95
19	12.66	0.55	800	13.33	10.66	8.88	7.62	6.66	5.33
20	12.05	0.59	842	12.05	9.64	8.03	6.88	6.03	4.82
21	11.47	0.66	885	10.92	8.74	7.28	6.24	5.46	4.37
22	10.95	0.73	927	9.95	7.96	6.64	5.69	4.98	3.98
23	10.48	0.79	969	9.11	7.29	6.07	5.21	4.56	3.65
24	10.04	0.86	1011	8.37	6.69	5.58	4.78	4.18	3.35
25	9.64	0.94	1053	7.71	6.17	5.14	4.41	3.86	3.09
26	9.27	1.01	1095	7.12	5.70	4.75	4.07	3.56	2.85
27	8.92	1.09	1137	6.61	5.28	4.40	3.78	3.31	2.64
28	8.61	1.18	1179	6.15	4.92	4.10	3.52	3.08	2.46
29	8.31	1.27	1222	5.73	4.58	3.82	3.27	2.87	2.29
30	8.03	1.36	1264	5.35	4.28	3.57	3.06	2.68	2.14
31	7.77	1.46	1306	5.01	4.01	3.34	2.86	2.50	2.00
32	7.53	1.55	1348	4.70	3.76	3.14	2.69	2.35	1.83
33	7.30	1.64	1390	4.42	3.54	2.95	2.53	2.21	1.77

STEEL I BEAMS.

12″ I BEAM. SHAPE No. 7. 101 LBS. PER YARD.

Depth, 12″. Width of flange, $4\frac{7}{16}$″. Thickness of web, $\frac{7}{16}$″.

$$\text{Safe load in nett tons} = \frac{187.20}{\text{Span in feet}}.$$

Maximum shear = 14.18 tons.

Span limit for uniformly distributed load of twice the maximum shear = 6.60′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	18.72	0.15	337	37.44	29.95	24.96	21.39	18.72	14.98
11	17.02	0.18	370	30.95	24.76	20.63	17.69	15.47	12.38
12	15.60	0.22	404	26.00	20.80	17.33	14.86	13.00	10.40
13	14.40	0.26	438	22.15	17.72	14.77	12.66	11.07	8.86
14	13.37	0.30	471	19.10	15.28	12.73	10.91	9.55	7.64
15	12.48	0.34	505	16.64	13.31	11.09	9.51	8.32	6.66
16	11.70	0.39	539	14.63	11.70	9.75	8.36	7.31	5.85
17	11.01	0.44	572	12.95	10.36	8.63	7.40	6.47	5.18
18	10.40	0.49	606	11.56	9.25	7.71	6.61	5.78	4.62
19	9.85	0.55	640	10.37	8.30	6.91	5.93	5.18	4.15
20	9.36	0.59	673	9.36	7.49	6.24	5.35	4.68	3.74
21	8.91	0.66	707	8.48	6.78	5.65	4.85	4.24	3.39
22	8.51	0.73	741	7.74	6.19	5.16	4.42	3.87	3.10
23	8.14	0.79	774	7.08	5.66	4.72	4.05	3.54	2.83
24	7.80	0.86	808	6.50	5.20	4.33	3.71	3.25	2.60
25	7.49	0.94	842	5.99	4.79	3.99	3.42	2.99	2.40
26	7.20	1.01	875	5.54	4.43	3.69	3.17	2.77	2.22
27	6.93	1.09	909	5.13	4.10	3.42	2.93	2.56	2.05
28	6.69	1.18	944	4.78	3.82	3.19	2.73	2.39	
29	6.46	1.27	977	4.46	3.57	2.97	2.55	2.23	
30	6.24	1.36	1010	4.16	3.33	2.77	2.38	2.08	
31	6.04	1.46	1044	3.90	3.12	2.60	2.23		
32	5.85	1.55	1077	3.66	2.93	2.44	2.09		
33	5.67	1.64	1111	3.44	2.75	2.29	1.97		

STEEL I BEAMS.

10½″ I BEAM. SHAPE No. 8. 136½ LBS. PER YARD.

Depth, 10½″. Width of flange, 5″. Thickness of web, $\frac{17}{32}$″.

$$\text{Safe load in nett tons} = \frac{236.70}{\text{Span in feet}}.$$

Maximum shear = 17.69 tons.

Span limit for uniformly distributed load of twice the maximum shear = 6.69′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	23.67	0.18	455	47.34	37.87	31.56	27.05	23.67	18.93
11	21.52	0.21	500	39.12	31.30	26.08	22.39	19.56	15.65
12	19.72	0.25	546	32.87	26.29	21.91	18.78	16.43	13.15
13	18.21	0.30	591	28.02	22.41	18.68	16.01	14.01	11.21
14	16.91	0.35	637	24.16	19.33	16.10	13.80	12.08	9.66
15	15.78	0.40	682	21.04	16.84	14.03	12.02	10.52	8.42
16	14.80	0.46	728	18.50	14.80	12.33	10.57	9.25	7.40
17	13.92	0.51	773	16.36	13.10	10.92	9.36	8.19	6.55
18	13.15	0.57	819	14.61	11.69	9.74	8.35	7.30	5.84
19	12.46	0.64	864	13.12	10.49	8.74	7.49	6.56	5.24
20	11.84	0.70	910	11.84	9.47	7.89	6.74	5.92	4.74
21	11.27	0.78	955	10.73	8.59	7.16	6.13	5.37	4.29
22	10.76	0.86	1001	9.78	7.83	6.52	5.59	4.88	3.91
23	10.30	0.94	1046	8.95	7.17	5.97	5.11	4.48	3.59
24	9.86	1.01	1092	8.22	6.57	5.48	4.70	4.11	3.29
25	9.47	1.11	1137	7.58	6.06	5.05	4.33	3.79	3.03
26	9.10	1.20	1183	7.00	5.60	4.67	4.00	3.50	2.80
27	8.77	1.29	1228	6.50	5.20	4.33	3.71	3.25	2.60
28	8.46	1.39	1274	6.04	4.83	4.03	3.45	3.02	2.42
29	8.16	1.48	1319	5.63	4.50	3.75	3.22	2.81	2.25
30	7.89	1.59	1365	5.26	4.21	3.51	3.01	2.63	2.10
31	7.64	1.69	1410	4.93	3.94	3.29	2.82	2.47	1.92
32	7.40	1.81	1456	4.63	3.70	3.09	2.64	2.31	1.85
33	7.18	1.92	1501	4.35	3.48	2.90	2.49	2.18	1.74

STEEL I BEAMS.

10½″ I BEAM. SHAPE No. 9. 106 LBS. PER YARD.

Depth, 10½″. Width of flange, $4\frac{7}{16}$″. Thickness of web, ½″.

Safe load in nett tons $= \dfrac{174.30}{\text{Span in feet}}$.

Maximum shear = 16.17 tons.

Span limit for uniformly distributed load of twice the maximum shear = 5.39′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	17.43	0.18	354	34.86	27.89	23.24	19.92	17.43	13.94
11	15.85	0.21	389	28.81	23.05	19.21	16.47	14.41	11.52
12	14.53	0.25	425	24.22	19.38	16.15	13.84	12.11	9.69
13	13.41	0.30	460	20.63	16.50	13.75	11.80	10.32	8.25
14	12.45	0.35	495	17.80	14.24	11.86	10.16	8.90	7.12
15	11.62	0.40	531	15.49	12.39	10.33	8.85	7.75	6.20
16	10.90	0.46	566	13.63	10.90	9.09	7.79	6.81	5.45
17	10.25	0.51	602	12.06	9.65	8.04	6.89	6.03	4.88
18	9.68	0.57	637	10.76	8.60	7.17	6.15	5.38	4.30
19	9.18	0.64	672	9.66	7.73	6.44	5.52	4.83	3.86
20	8.72	0.70	708	8.72	6.98	5.81	4.98	4.36	3.49
21	8.30	0.78	743	7.90	6.32	5.27	4.52	3.95	3.16
22	7.92	0.86	778	7.20	5.76	4.80	4.10	3.60	2.88
23	7.58	0.94	814	6.59	5.27	4.40	3.77	3.30	2.64
24	7.27	1.01	849	6.06	4.84	4.04	3.46	3.03	2.42
25	6.97	1.11	885	5.58	4.46	3.72	3.19	2.79	2.23
26	6.71	1.20	920	5.16	4.13	3.44	2.95	2.58	2.07
27	6.46	1.29	955	4.78	3.83	3.19	2.73	2.39	1.92
28	6.23	1.39	991	4.45	3.56	2.97	2.54	2.23	1.78
29	6.01	1.48	1026	4.15	3.32	2.76	2.37	2.07	1.66
30	5.81	1.59	1061	3.87	3.10	2.58	2.21	1.94	1.55
31	5.62	1.69	1097	3.63	2.90	2.42	2.07	1.81	1.45
32	5.45	1.81	1132	3.41	2.73	2.27	1.95	1.74	1.36
33	5.28	1.92	1168	3.20	2.56	2.14	1.83	1.60	1.28

STEEL I BEAMS.

10½″ I BEAM. SHAPE No. 10. 91 LBS. PER YARD.

Depth, 10½″. Width of flange, 4⅛″. Thickness of web, $\frac{13}{32}$″.

$$\text{Safe load in nett tons} = \frac{149.60}{\text{Span in feet}}.$$

Maximum shear = 12.10 tons.

Span limit for uniformly distributed load of twice the maximum shear = 6.14′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	14.96	0.18	303	29.92	23.94	19.95	17.10	14.96	11.97
11	13.59	0.21	334	24.71	19.77	16.47	14.12	12.35	9.88
12	12.46	0.25	364	20.77	16.62	13.85	11.87	10.38	8.31
13	11.50	0.30	394	17.70	14.16	11.80	10.11	8.85	7.08
14	10.68	0.35	425	15.26	12.21	10.17	8.72	7.63	6.10
15	9.97	0.40	455	13.30	10.64	8.87	7.60	6.65	5.32
16	9.35	0.46	485	11.70	9.35	7.80	6.68	5.85	4.68
17	8.80	0.51	516	10.35	8.28	6.90	5.92	5.18	4.14
18	8.31	0.57	546	9.23	7.38	6.15	5.27	4.62	3.70
19	7.87	0.64	576	8.28	6.63	5.52	4.73	4.14	3.32
20	7.48	0.70	607	7.48	5.98	4.99	4.27	3.74	2.94
21	7.12	0.78	637	6.78	5.42	4.52	3.88	3.39	2.71
22	6.80	0.86	667	6.18	4.94	4.12	3.54	3.09	2.47
23	6.51	0.94	698	5.66	4.53	3.78	3.23	2.83	2.26
24	6.23	1.01	728	5.19	4.15	3.46	2.97	2.60	2.08
25	5.98	1.11	758	4.78	3.83	3.19	2.73	2.39	
26	5.75	1.20	789	4.42	3.54	2.95	2.53	2.21	
27	5.54	1.29	819	4.10	3.28	2.74	2.35	2.05	
28	5.34	1.39	849	3.81	3.05	2.54	2.17		
29	5.16	1.48	880	3.56	2.85	2.37	2.03		
30	4.99	1.59	910	3.33	2.66	2.22			
31	4.82	1.69	940	3.11	2.49	2.07			
32	4.67	1.81	971	2.92	2.34				
33	4.54	1.92	1001	2.75	2.20				

STEEL I BEAMS.

10″ I BEAM. SHAPE No. 11. 106 LBS. PER YARD.

Depth, 10″. Width of flange, 4⅝″. Thickness of web, ½″.

$$\text{Safe load in nett tons} = \frac{167.70}{\text{Span in feet}}.$$

Maximum shear = 15.85 tons.

Span limit for uniformly distributed load of twice the maximum shear = 5.29′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	16.77	0.20	353	33.54	26.83	22.36	19.17	16.77	13.42
11	15.24	0.23	389	27.71	22.17	18.47	15.83	13.86	11.08
12	13.98	0.27	424	23.30	18.64	15.53	13.31	11.65	9.32
13	12.90	0.31	459	19.85	15.88	13.23	11.34	9.93	7.94
14	11.98	0.36	495	17.11	13.69	11.41	9.80	8.56	6.84
15	11.18	0.43	530	14.91	11.93	9.94	8.52	7.46	5.96
16	10.48	0.48	566	13.10	10.48	8.73	7.49	6.55	5.24
17	9.87	0.53	601	11.61	9.29	7.74	6.64	5.81	4.64
18	9.32	0.60	636	10.31	8.25	6.87	5.89	5.16	4.12
19	8.83	0.68	671	9.26	7.41	6.17	5.29	4.63	3.70
20	8.39	0.75	707	8.39	6.71	5.59	4.79	4.20	3.36
21	7.99	0.83	742	7.61	6.09	5.07	4.35	3.81	3.04
22	7.62	0.91	777	6.93	5.54	4.62	3.96	3.47	2.77
23	7.29	0.99	813	6.34	5.07	4.23	3.63	3.17	2.54
24	6.99	1.08	848	5.83	4.66	3.89	3.33	2.92	2.33
25	6.71	1.18	883	5.37	4.30	3.58	3.07	2.68	2.15
26	6.45	1.27	919	4.96	3.97	3.31	2.83	2.48	1.99
27	6.21	1.36	954	4.60	3.68	3.07	2.63	2.30	
28	5.99	1.47	989	4.28	3.42	2.85	2.44	2.14	
29	5.78	1.57	1025	4.00	3.20	2.67	2.29	2.00	
30	5.59	1.68	1060	3.73	2.98	2.49	2.13		
31	5.41	1.79	1095	3.49	2.79	2.33	2.00		
32	5.24	1.92	1131	3.28	2.62	2.19			
33	5.08	2.04	1166	3.08	2.46	2.05			

STEEL I BEAMS.

10″ I BEAM. SHAPE No. 12. 91 LBS. PER YARD.

Depth, 10″. Width of flange, 4⅜″. Thickness of web, $\frac{7}{16}$″.

$$\text{Safe load in nett tons} = \frac{145.60}{\text{Span in feet}}.$$

Maximum shear = 13.05 tons.

Span limit for uniformly distributed load of twice the maximum shear = 5.58′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	14.56	0.20	303	29.12	23.30	19.41	16.64	14.56	11.65
11	13.24	0.23	334	24.07	19.26	16.05	13.75	12.03	9.63
12	12.13	0.27	364	20.20	16.16	13.47	11.54	10.10	8.08
13	11.20	0.31	394	17.23	13.78	11.49	9.85	8.61	6.89
14	10.40	0.36	425	14.86	11.89	9.91	8.49	7.43	5.94
15	9.71	0.43	455	12.95	10.36	8.63	7.40	6.47	5.18
16	9.10	0.48	485	11.38	9.10	7.59	6.50	5.69	4.55
17	8.56	0.53	516	10.07	8.06	6.71	5.75	5.03	4.03
18	8.09	0.60	546	8.99	7.19	5.99	5.14	4.49	3.60
19	7.66	0.68	576	8.06	6.45	5.37	4.61	4.03	3.22
20	7.28	0.75	607	7.28	5.82	4.85	4.16	3.64	2.91
21	6.93	0.83	637	6.60	5.28	4.40	3.77	3.30	2.64
22	6.62	0.91	667	6.02	4.82	4.01	3.44	3.00	2.41
23	6.33	0.99	698	5.50	4.40	3.67	3.14	2.75	2.20
24	6.07	1.08	728	5.06	4.05	3.37	2.89	2.53	2.02
25	5.82	1.18	758	4.66	3.73	3.11	2.66	2.33	
26	5.60	1.27	789	4.31	3.45	2.87	2.46	2.15	
27	5.39	1.36	819	3.99	3.19	2.66	2.28		
28	5.20	1.47	849	3.71	2.97	2.47	2.12		
29	5.02	1.57	880	3.46	2.77	2.31			
30	4.85	1.68	910	3.23	2.58	2.15			
31	4.69	1.79	940	3.03	2.42	2.02			
32	4.55	1.92	971	2.84	2.27				
33	4.41	2.04	1001	2.67	2.14				

STEEL I BEAMS.

9″ I BEAM. SHAPE No. 13. 91 LBS. PER YARD.

Depth, 9″. Width of flange, 4⅜″. Thickness of web, ½″.

$$\text{Safe load in nett tons} = \frac{127.40}{\text{Span in feet}}.$$

Maximum shear = 14.90 tons.

Span limit for uniformly distributed load of twice the maximum shear = 4.28′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	12.74	0.20	303	25.48	20.38	16.99	14.56	12.74	10.19
11	11.58	0.25	334	21.05	16.84	14.03	12.03	10.52	8.42
12	10.62	0.30	364	17.70	14.16	11.80	10.11	8.85	7.08
13	9.80	0.35	394	15.08	12.06	10.05	8.62	7.54	6.03
14	9.10	0.40	425	13.00	10.40	8.67	7.43	6.50	5.20
15	8.49	0.46	455	11.32	9.06	7.55	6.47	5.66	4.53
16	7.96	0.52	485	9.95	7.96	6.63	5.69	4.97	3.98
17	7.49	0.60	516	8.81	7.05	5.87	5.03	4.40	3.52
18	7.08	0.66	546	7.88	6.30	5.25	4.50	3.94	3.15
19	6.70	0.74	576	7.05	5.64	4.70	4.03	3.52	2.82
20	6.37	0.82	607	6.37	5.10	4.25	3.64	3.18	2.55
21	6.07	0.91	637	5.78	4.62	3.85	3.30	2.89	2.31
22	5.79	1.00	667	5.26	4.21	3.51	3.01	2.63	2.10
23	5.54	1.09	698	4.82	3.86	3.21	2.75	2.41	
24	5.31	1.18	728	4.43	3.54	2.95	2.53	2.21	
25	5.10	1.29	758	4.08	3.26	2.72	2.33	2.04	
26	4.90	1.39	789	3.77	3.02	2.51	2.15		
27	4.72	1.51	819	3.50	2.80	2.33	2.00		
28	4.55	1.61	849	3.25	2.60	2.17			
29	4.39	1.73	880	3.03	2.42				
30	4.25	1.86	910	2.83	2.26				
31	4.11	1.99	940	2.65	2.12				
32	3.98	2.12	971	2.49					
33	3.86	2.26	1001	2.34					

STEEL I BEAMS.

9″ I BEAM. SHAPE No. 14. 86 LBS. PER YARD.

Depth, 9″. Width of flange, 4¼″. Thickness of web, $\frac{7}{16}$″.

$$\text{Safe load in nett tons} = \frac{124.20}{\text{Span in feet}}.$$

Maximum shear = 12.29 tons.

Span limit for uniformly distributed load of twice the maximum shear = 5.05′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	12.42	0.20	286	24.84	19.87	16.56	14.20	12.42	9.93
11	11.29	0.25	315	20.53	16.42	13.70	11.73	10.27	8.21
12	10.35	0.30	344	17.25	13.80	11.50	9.86	8.62	6.90
13	9.55	0.35	372	14.70	11.76	9.80	8.40	7.35	5.88
14	8.87	0.40	401	12.67	10.14	8.45	7.24	6.34	5.07
15	8.28	0.46	430	11.04	8.83	7.36	6.31	5.52	4.41
16	7.76	0.52	458	9.70	7.76	6.47	5.54	4.85	3.88
17	7.31	0.60	487	8.60	6.88	5.73	4.92	4.30	3.44
18	6.85	0.66	516	7.61	6.09	5.07	4.35	3.81	3.05
19	6.54	0.74	544	6.88	5.50	4.59	3.93	3.44	2.75
20	6.21	0.82	573	6.21	4.98	4.14	3.55	3.11	2.49
21	5.92	0.91	601	5.64	4.51	3.76	3.22	2.82	2.25
22	5.65	1.00	630	5.14	4.11	3.43	2.94	2.57	2.06
23	5.40	1.09	659	4.70	3.76	3.13	2.68	2.35	
24	5.18	1.18	687	4.32	3.45	2.88	2.47	2.16	
25	4.97	1.29	716	3.98	3.18	2.65	2.27		
26	4.78	1.39	745	3.68	2.94	2.45	2.10		
27	4.60	1.51	773	3.41	2.73	2.27			
28	4.44	1.61	802	3.17	2.54	2.11			
29	4.28	1.73	831	2.95	2.36				
30	4.14	1.86	859	2.76	2.21				
31	4.00	1.99	888	2.58	2.06				
32	3.88	2.12	917	2.43					
33	3.77	2.26	945	2.28					

STEEL I BEAMS.

9″ I BEAM. SHAPE No. 15. 70¾ LBS. PER YARD.

Depth, 9″. Width of flange, 4″. Thickness of web, ⅜″.

$$\text{Safe load in nett tons} = \frac{96.20}{\text{Span in feet}}.$$

Maximum shear = 9.77 tons.

Span limit for uniformly distributed load of twice the maximum shear = 4.93′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	9.62	0.20	236	19.24	15.39	12.83	10.99	9.62	7.70
11	8.74	0.25	259	15.89	12.71	10.59	9.08	7.94	6.36
12	8.02	0.30	283	13.37	10.70	8.91	7.64	6.68	5.35
13	7.40	0.35	307	11.38	9.10	7.59	6.50	5.69	4.55
14	6.88	0.40	339	9.83	7.86	6.55	5.62	4.91	3.93
15	6.41	0.46	354	8.55	6.84	5.70	4.89	4.27	3.42
16	6.01	0.52	378	7.51	6.01	5.01	4.29	3.75	3.00
17	5.66	0.60	401	6.66	5.33	4.44	3.81	3.33	2.66
18	5.34	0.66	424	5.93	4.74	3.95	3.39	2.96	2.37
19	5.06	0.74	448	5.33	4.26	3.55	3.05	2.66	2.13
20	4.81	0.82	471	4.81	3.85	3.21	2.75	2.41	
21	4.58	0.91	495	4.36	3.49	2.91	2.49	2.18	
22	4.37	1.00	519	3.97	3.18	2.65	2.27		
23	4.18	1.09	542	3.63	2.90	2.42	2.07		
24	4.01	1.18	566	3.34	2.67	2.23			
25	3.85	1.29	590	3.08	2.46	2.05			
26	3.70	1.39	613	2.85	2.28				
27	3.56	1.51	637	2.64	2.11				
28	3.44	1.61	661	2.46					
29	3.32	1.73	684	2.29					
30	3.21	1.86	708	2.14					
31	3.10	1.99	732	2.00					
32	3.00	2.12	755						
33	2.92	2.26	778						

STEEL I BEAMS.

8″ I BEAM. SHAPE No. 16. 81 LBS. PER YARD.

Depth, 8″. Width of flange, $4\frac{5}{32}$″. Thickness of web, ½″.

$$\text{Safe load in nett tons} = \frac{100.10}{\text{Span in feet}}.$$

Maximum shear = 13.60 tons.

Span limit for uniformly distributed load of twice the maximum shear = 3.68′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	10.01	0.23	270	20.02	16.02	13.35	11.44	10.01	8.01
11	9.10	0.29	297	16.54	13.23	11.00	9.45	8.27	6.62
12	8.34	0.34	324	13.90	11.12	9.27	7.95	6.95	5.56
13	7.70	0.39	350	11.85	9.48	7.90	6.77	5.93	4.74
14	7.15	0.46	377	10.21	8.17	6.81	5.83	5.11	4.08
15	6.67	0.52	404	8.89	7.11	5.93	5.08	4.45	3.55
16	6.26	0.60	431	7.82	6.26	5.21	4.47	3.91	3.13
17	5.89	0.68	458	6.93	5.54	4.62	3.96	3.47	2.77
18	5.56	0.75	485	6.18	4.94	4.12	3.53	3.09	2.47
19	5.27	0.83	512	5.55	4.44	3.70	3.17	2.78	2.22
20	5.00	0.92	539	5.00	4.00	3.33	2.86	2.50	2.00
21	4.77	1.02	566	4.54	3.63	3.03	2.59	2.27	
22	4.55	1.12	593	4.14	3.31	2.76	2.37	2.07	
23	4.35	1.22	620	3.78	3.02	2.52	2.16		
24	4.17	1.34	647	3.47	2.78	2.31			
25	4.00	1.46	674	3.20	2.56	2.13			
26	3.85	1.56	701	2.96	2.37				
27	3.70	1.69	728	2.74	2.19				
28	3.57	1.82	755	2.55	2.04				
29	3.45	1.95	782	2.38					
30	3.34	2.08	809	2.23					
31	3.23	2.22	836	2.08					
32	3.13	2.36	863						
33	3.03	2.51	890						

STEEL I BEAMS.

8″ I BEAM. SHAPE No. 17. 65¾ LBS. PER YARD.

Depth, 8″. Width of flange, 4″. Thickness of web, $\frac{5}{16}$″.

$$\text{Safe load in nett tons} = \frac{88.40}{\text{Span in feet}}.$$

Maximum shear = 6.97 tons.

Span limit for uniformly distributed load of twice the maximum shear = 6.34′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	8.84	0.23	219	17.68	14.14	11.79	10.10	8.84	7.07
11	8.04	0.29	241	14.62	11.69	9.75	8.35	7.31	5.85
12	7.37	0.34	263	12.28	9.82	8.19	7.02	6.14	4.91
13	6.80	0.39	285	10.46	8.37	6.97	5.98	5.23	4.19
14	6.31	0.46	307	9.01	7.21	6.01	5.15	4.51	3.61
15	5.89	0.52	329	7.85	6.28	5.23	4.49	3.93	3.14
16	5.53	0.60	350	6.91	5.53	4.61	3.95	3.46	2.77
17	5.20	0.68	372	6.12	4.89	4.08	3.49	3.06	2.45
18	4.91	0.75	394	5.46	4.37	3.64	3.12	2.73	2.19
19	4.65	0.83	416	4.89	3.91	3.26	2.79	2.45	
20	4.42	0.92	438	4.42	3.54	2.95	2.53	2.21	
21	4.21	1.02	460	4.01	3.21	2.67	2.29	2.01	
22	4.02	1.12	482	3.65	2.92	2.40	2.09		
23	3.84	1.22	504	3.34	2.67	2.23			
24	3.68	1.34	526	3.07	2.46	2.05			
25	3.54	1.46	548	2.83	2.26				
26	3.40	1.56	569	2.62	2.09				
27	3.27	1.69	591	2.42					
28	3.16	1.82	613	2.26					
29	3.05	1.95	635	2.10					
30	2.95	2.08	657						
31	2.85	2.22	679						
32	2.76	2.36	701						
33	2.68	2.51	723						

STEEL I BEAMS.

7″ I BEAM. SHAPE No. 18. 65¾ LBS. PER YARD.

Depth, 7″. Width of flange, $3\frac{9}{16}$″. Thickness of web, $\frac{29}{64}$″.

Safe load in nett tons $= \dfrac{71.50}{\text{Span in feet}}$.

Maximum shear = 10.90 tons.

Span limit for uniformly distributed load of twice the maximum shear = 3.28′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	7.15	0.26	219	14.30	11.44	9.53	8.17	7.15	5.72
11	6.50	0.33	241	11.82	9.46	7.88	6.75	5.91	4.73
12	5.96	0.38	263	9.93	7.94	6.62	5.67	4.97	3.97
13	5.50	0.46	285	8.46	6.77	5.64	4.83	4.23	3.38
14	5.11	0.52	307	7.30	5.84	4.87	4.17	3.65	2.92
15	4.77	0.60	329	6.36	5.09	4.24	3.63	3.18	2.54
16	4.47	0.68	351	5.59	4.47	3.73	3.20	2.29	2.23
17	4.21	0.77	373	4.96	3.97	3.31	2.83	2.48	
18	3.97	0.86	394	4.41	3.53	2.94	2.52	2.20	
19	3.76	0.96	416	3.96	3.17	2.64	2.26		
20	3.58	1.07	438	3.58	2.86	2.39	2.05		
21	3.41	1.17	460	3.25	2.60	2.17			
22	3.25	1.29	482	2.95	2.36				
23	3.11	1.40	504	2.70	2.16				
24	2.98	1.52	526	2.48					
25	2.86	1.65	548	2.29					
26	2.75	1.79	570	2.12					
27	2.65	1.94	592						
28	2.55	2.08	614						
29	2.46	2.24	636						
30	2.38	2.39	657						
31	2.31	2.55	679						
32	2.23	2.70	701						
33	2.16	2.86	723						

Span limit for tabular safe loads = 9.00′.

STEEL I BEAMS.

7″ I BEAM. SHAPE No. 19. 55½ LBS. PER YARD.

Depth, 7″. Width of flange, $3\frac{7}{16}$″. Thickness of web, $\frac{21}{64}$″.

$$\text{Safe load in nett tons} = \frac{65.40}{\text{Span in feet}}.$$

Maximum shear = 7.08 tons.

Span limit for uniformly distributed load of twice the maximum shear = 4.62′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	6.53	0.26	185	13.06	10.45	8.71	7.46	6.53	5.23
11	5.94	0.33	204	10.80	8.64	7.20	6.17	5.40	4.32
12	5.45	0.38	222	9.08	7.26	6.05	5.19	4.54	3.63
13	5.03	0.46	241	7.74	6.19	5.16	4.42	3.87	3.10
14	4.67	0.52	259	6.67	5.34	4.45	3.81	3.34	2.67
15	4.36	0.60	278	5.81	4.65	3.87	3.32	2.91	2.32
16	4.09	0.68	297	5.11	4.09	3.41	2.92	2.56	2.04
17	3.85	0.77	315	4.53	3.62	3.02	2.59	2.27	
18	3.63	0.86	334	4.03	3.22	2.69	2.30	2.02	
19	3.44	0.96	352	3.62	2.89	2.41	2.07		
20	3.27	1.07	371	3.27	2.62	2.18			
21	3.11	1.17	389	2.96	2.37				
22	2.97	1.29	408	2.70	2.16				
23	2.84	1.40	426	2.47					
24	2.72	1.52	445	2.27					
25	2.61	1.65	463	2.09					
26	2.51	1.79	482						
27	2.42	1.94	500						
28	2.33	2.08	519						
29	2.25	2.24	537						
30	2.18	2.39	556						
31	2.11	2.55	574						
32	2.04	2.70	593						
33	1.98	2.86	612						

Span limit for tabular safe load = 8½′.

STEEL I BEAMS.

6″ I BEAM. SHAPE No. 20. 50½ LBS. PER YARD.

Depth, 6″. Width of flange, $3\frac{9}{32}$″. Thickness of web, $\frac{13}{32}$″.

$$\text{Safe load in nett tons} = \frac{46.80}{\text{Span in feet}}.$$

Maximum shear = 8.52 tons.

Span limit for uniformly distributed load of twice the maximum shear = 2.75′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	4.68	0.33	168	9.36	7.49	6.24	5.35	4.68	3.74
11	4.25	0.38	185	7.61	6.09	5.07	4.35	3.80	3.04
12	3.90	0.44	202	6.50	5.20	4.33	3.71	3.25	2.60
13	3.60	0.52	219	5.54	4.43	3.69	3.16	2.77	2.21
14	3.34	0.61	236	4.77	3.82	3.18	2.72	2.38	1.91
15	3.12	0.70	253	4.16	3.33	2.77	2.38	2.08	
16	2.92	0.78	269	3.65	2.92	2.43	2.09		
17	2.75	0.89	286	3.23	2.58	2.15			
18	2.60	1.00	303	2.88	2.30				
19	2.46	1.12	320	2.59	2.07				
20	2.34	1.23	337	2.34					
21	2.23	1.36	353	2.12					
22	2.13	1.49	370						
23	2.04	1.64	387						
24	1.95	1.78	404						
25	1.87	1.94	421						
26	1.80	2.09	438						
27	1.74	2.26	454						
28	1.67	2.43	471						
29	1.61	2.60	488						
30	1.56	2.78	505						
31	1.51	2.95	522						
32	1.46	3.12	539						
33	1.42	3.29	555						

Span limit for tabular safe load = 8.10′.

STEEL I BEAMS.

6″ I BEAM. SHAPE No. 21. 40½ LBS. PER YARD.

Depth, 6″. Width of flange, 3⅛″. Thickness of web, ¼″.

$$\text{Safe load in nett tons} = \frac{41.60}{\text{Span in feet}}.$$

Maximum shear = 4.41 tons.

Span limit for uniformly distributed load of twice the maximum shear = 4.72′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
10	4.16	0.33	135	8.32	6.66	5.55	4.75	4.16	3.33
11	3.78	0.38	148	6.87	5.50	4.58	3.93	3.44	2.75
12	3.47	0.44	162	5.78	4.62	3.85	3.30	2.89	2.31
13	3.20	0.52	175	4.92	3.94	3.28	2.81	2.46	
14	2.97	0.61	189	4.24	3.39	2.83	2.42	2.12	
15	2.77	0.70	202	3.69	2.95	2.46	2.11		
16	2.60	0.78	216	3.25	2.60	2.17			
17	2.45	0.89	229	2.88	2.30				
18	2.31	1.00	243	2.57	2.06				
19	2.19	1.12	256	2.30					
20	2.08	1.23	270	2.08					
21	1.98	1.36	283						
22	1.89	1.49	297						
23	1.81	1.64	310						
24	1.73	1.78	324						
25	1.66	1.94	337						
26	1.60	2.09	350						
27	1.54	2.26	364						
28	1.49	2.43	377						
29	1.43	2.60	391						
30	1.39	2.78	404						
31	1.34	2.95	418						
32	1.30	3.12	431						
33	1.26	3.29	445						

Span limit for tabular safe load = 8½′.

STEEL I BEAMS.

5″ I BEAM. SHAPE No. 22. 40½ LBS. PER YARD.

Depth, 5″. Width of flange, $2\frac{15}{16}$″. Thickness of web, ⅜″.

$$\text{Safe load in nett tons} = \frac{33.30}{\text{Span in feet}}.$$

Maximum shear = 6.71 tons.

Span limit for uniformly distributed load of twice the maximum shear = 2.48′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
3	11.09	0.03	40						29.57
4	8.32	0.05	54			27.73	23.77	20.80	16.64
5	6.65	0.09	68	26.60	21.28	17.73	15.20	13.30	10.64
6	5.55	0.13	80	18.50	14.80	12.33	10.57	9.25	7.40
7	4.75	0.18	94	13.57	10.86	9.05	7.75	6.79	5.43
8	4.16	0.23	108	10.40	8.32	6.93	5.94	5.20	4.16
9	3.69	0.30	120	8.20	6.56	5.47	4.69	4.10	3.28
10	3.33	0.36	135	6.66	5.33	4.44	3.81	3.33	2.67
11	3.03	0.44	148	5.51	4.41	3.67	3.15	2.76	2.21
12	2.77	0.53	162	4.62	3.70	3.08	2.64	2.31	
13	2.56	0.62	175	3.94	3.15	2.63	2.25		
14	2.38	0.73	189	3.40	2.72	2.27			
15	2.21	0.83	202	2.95	2.36				
16	2.08	0.95	216	2.60	2.08				
17	1.95	1.07	229	2.29					
18	1.85	1.19	242	2.06					
19	1 75	1.34	256						
20	1.67	1.48	269						
21	1.58	1.64	283						
22	1.51	1.79	296						
23	1.45	1.96	310						
24	1.38	2.14	322						
25	1.33	2.33	337						
26	1.28	2.53	350						

STEEL I BEAMS.

5″ I BEAM. SHAPE No. 23. 30½ LBS. PER YARD.

Depth, 5″. Width of flange, 2¾″. Thickness of web, $\frac{3}{16}$″.

$$\text{Safe load in nett tons} = \frac{25.00}{\text{Span in feet}}.$$

Maximum shear = 2.54 tons.

Span limit for uniformly distributed load of twice the maximum shear = 5.00′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
3	8.32	0.03	30						22.19
4	6.24	0.05	40		24.96	20.80	17.83	15.60	12.48
5	4.99	0.09	51	19.96	15.97	13.31	11.41	9.98	7.99
6	4.16	0.13	61	13.87	11.10	9.25	7.93	6.94	5.55
7	3.57	0.18	71	10.20	8.16	6.80	5.83	5.10	4.08
8	3.12	0.23	81	7.80	6.24	5.20	4.46	3.90	3.12
9	2.77	0.30	91	6.16	4.93	4.11	3.52	3.08	2.47
10	2.50	0.36	101	5.00	4.00	3.33	2.86	2.50	2.00
11	2.27	0.44	111	4.13	3.30	2.75	2.36	2.07	
12	2.08	0.53	121	3.47	2.78	2.31			
13	1.92	0.62	131	2.95	2.36				
14	1.78	0.73	141	2.54	2.03				
15	1.66	0.83	152	2.21					
16	1.56	0.95	162						
17	1.47	1.07	172						
18	1.38	1.19	182						
19	1.31	1.34	192						
20	1.25	1.48	202						
21	1.19	1.64	212						
22	1.13	1.79	222						
23	1.09	1.96	233						
24	1.04	2.14	243						
25	0.99	2.33	253						
26	0.96	2.53	263						

STEEL I BEAMS.

4″ I BEAM. SHAPE No. 24. 30½ LBS. PER YARD.

Depth, 4″. Width of flange, $2\frac{7}{16}$″. Thickness of web, $\frac{27}{64}$″.

$$\text{Safe load in nett tons} = \frac{18.20}{\text{Span in feet}}.$$

Maximum shear = 6.32 tons.

Span limit for uniformly distributed load of twice the maximum shear = 1.44′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
3	6.07	0.04	30	40.47	32.38	26.98	23.13	20.24	16.19
4	4.55	0.08	40	22.75	18.20	15.17	13.00	11.38	9.10
5	3.64	0.12	51	14.56	11.65	9.71	8.32	7.28	5.83
6	3.03	0.17	61	10.10	8.08	6.73	5.77	5.05	4.04
7	2.60	0.22	71	7.43	5.94	4.95	4.25	3.72	2.97
8	2.28	0.30	81	5.70	4.56	3.80	3.26	2.85	2.28
9	2.02	0.38	91	4.49	3.59	2.99	2.57	2.25	
10	1.82	0.47	101	3.64	2.91	2.43	2.08		
11	1.65	0.56	111	3.00	2.40	2.00			
12	1.52	0.66	121	2.53	2.02				
13	1.40	0.78	131	2.15					
14	1.30	0.91	141						
15	1.21	1.05	152						
16	1.14	1.18	162						
17	1.07	1.34	172						
18	1.01	1.51	182						
19	0.96	1.68	192						
20	0.91	1.86	202						
21	0.87	2.05	212						
22	0.83	2.25	222						
23	0.79	2.46	233						
24	0.76	2.68	243						
25	0.73	2.90	253						
26	0.70	3.13	263						

STEEL I BEAMS.

4″ I BEAM. SHAPE No. 25. 24¼ LBS. PER YARD.

Depth, 4″. Width of flange, 2¼″. Thickness of web, $\frac{5}{16}$″.

$$\text{Safe load in nett tons} = \frac{14.56}{\text{Span in feet}}.$$

Maximum shear = 4.51 tons.

Span limit for uniformly distributed load of twice the maximum shear = ′1.61.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
3	4.85	0.04	24	32.33	25.86	21.55	18.47	16.17	12.93
4	3.64	0.08	32	18.20	14.56	12.13	10.40	9.10	7.28
5	2.91	0.12	40	11.64	9.31	7.76	6.65	5.82	4.66
6	2.43	0.17	48	8.10	6.48	5.40	4.60	4.05	3.24
7	2.08	0.22	56	5.94	4.75	3.96	3.38	2.97	2.38
8	1.82	0.30	64	4.55	3.64	3.03	2.60	2.28	
9	1.62	0.38	73	3.60	2.88	2.40	2.06		
10	1.46	0.47	81	2.92	2.34				
11	1.32	0.56	89	2.40					
12	1.21	0.66	97	2.02					
13	1.12	0.78	105						
14	1.04	0.91	113						
15	0.97	1.05	121						
16	0.91	1.18	129						
17	0.86	1.34	137						
18	0.81	1.51	146						
19	0.76	1.68	154						
20	0.73	1.86	162						
21	0.69	2.05	170						
22	0.66	2.25	178						
23	0.63	2.46	186						
24	0.60	2.68	194						
25	0.58	2.90	202						
26	0.56	3.13	210						

STEEL I BEAMS.

4″ I BEAM. SHAPE No. 26. 18¼ LBS. PER YARD.

Depth, 4″. Width of flange, 2⅛″. Thickness of web, $\frac{3}{16}$″.

$$\text{Safe load in nett tons} = \frac{11.40}{\text{Span in feet}}.$$

Maximum shear = 2.31 tons.

Span limit for uniformly distributed load of twice the maximum shear = 2.47′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
3	3.80	0.04	18	25.33	20.26	16.89	14.47	12.67	10.13
4	2.85	0.08	24	14.25	11.40	9.50	8.14	7.13	5.70
5	2.28	0.12	30	9.12	7.30	6.08	5.21	4.56	3.65
6	1.90	0.17	36	6.33	5.06	4.22	3.62	3.17	2.53
7	1.63	0.22	42	4.66	3.73	3.07	2.66	2.33	
8	1.43	0.30	48	3.58	2.86	2.39	2.05		
9	1.27	0.38	55	2.82	2.26				
10	1.14	0.47	61	2.28					
11	1.04	0.56	67						
12	0.95	0.66	73						
13	0.88	0.78	79						
14	0.81	0.91	85						
15	0.76	1.05	91						
16	0.71	1.18	97						
17	0.67	1.34	103						
18	0.63	1.51	109						
19	0.60	1.68	115						
20	0.57	1.86	121						
21	0.54	2.05	128						
22	0.52	2.25	134						
23	0.49	2.46	140						
24	0.48	2.68	146						
25	0.46	2.90	152						
26	0.44	3.13	158						

TABLES

OF THE CAPACITY OF

STEEL CHANNELS

UNDER UNIFORMLY DISTRIBUTED
TRANSVERSE LOADS,

THE EXTREME FIBRE STRESS BEING 7.8 TONS PER SQUARE INCH, WHICH IS TWO-SEVENTHS OF

THE MODULUS OF RUPTURE;

AND THE UNSTAYED LENGTH OF FLANGE NOT EXCEEDING THIRTY TIMES ITS WIDTH.

The span, which is *thirty* times the flange width, is denoted by a dotted line on the tables, and for lengths greater than this, the tabular safe load must be reduced by multiplying it by the factors given in table on page 43, or else some method of staying the flanges be employed.

STEEL CHANNELS.

15″ CHANNEL. SHAPE No. 30. 227¼ LBS. PER YARD.

Depth, 15″. Width of flange, $5\frac{5}{64}$″. Thickness of web, $1\frac{5}{64}$″.

$$\text{Safe load in nett tons} = \frac{431.60}{\text{Span in feet}}.$$

Maximum shear = 57.08 tons.

Span limit for uniformly distributed load of twice the maximum shear = 3.78′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	71.93	0.04	455						
8	53.95	0.09	606						
10	43.16	0.14	758					43.16	34.53
12	35.97	0.19	909			39.97	34.26	29.97	23.98
14	30.83	0.27	1061	44.04	35.23	29.36	25.17	22.02	17.62
16	26.98	0.35	1212	33.73	26.98	22.49	19.27	16.86	13.49
18	23.97	0.44	1364	26.63	21.30	17.75	15.22	13.31	10.65
20	21.58	0.56	1515	21.58	17.26	14.39	12.33	10.79	8.63
22	19.62	0.68	1667	17.84	14.27	11.89	10.19	8.92	7.14
24	17.98	0.81	1818	14.98	11.98	9.99	8.56	7.49	5.99
26	16.60	0.95	1970	12.77	10.22	8.51	7.30	6.38	5.11
28	15.42	1.09	2121	11.01	8.81	7.34	6.29	5.50	4.40
30	14.39	1.25	2273	9.59	7.67	6.39	5.48	4.79	3.84
32	13.49	1.43	2424	8.43	6.74	5.62	4.82	4.21	3.37
34	12.69	1.62	2576	7.46	5.97	4.97	4.26	3.73	2.98

STEEL CHANNELS.

15″ CHANNEL. SHAPE No. 30. 176¾ LBS. PER YARD.

Depth, 15″. Width of flange, 4¾″. Thickness of web, ¾″.

$$\text{Safe load in nett tons} = \frac{365.30}{\text{Span in feet}}.$$

Maximum shear = 35.66 tons.

Span limit for uniformly distributed load of twice the maximum shear = 5.01′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	60.88	0.04	354						
8	45.66	0.09	471						45.66
10	36.53	0.14	589				41.75	36.53	29.22
12	30.44	0.19	707		40.58	33.82	28.99	25.36	20.29
14	26.09	0.27	825	37.27	29.82	24.85	21.30	18.63	14.91
16	22.83	0.35	943	28.54	22.83	19.03	16.31	14.27	11.42
18	20.29	0.44	1060	22.54	18.03	15.03	12.88	11.27	9.02
20	18.27	0.56	1178	18.27	14.62	12.18	10.44	9.13	7.31
22	16.60	0.68	1296	15.09	12.07	10.06	8.62	7.54	6.04
24	15.22	0.81	1414	12.68	10.14	8.45	7.25	6.34	5.07
26	14.05	0.95	1532	10.81	8.65	7.21	6.18	5.40	4.32
28	13.05	1.09	1650	9.32	7.46	6.21	5.33	4.66	3.73
30	12.17	1.25	1767	8.11	6.49	5.41	4.63	4.05	3.24
32	11.42	1.43	1885	7.14	5.71	4.76	4.08	3.57	2.86
34	10.74	1.62	2003	6.32	5.06	4.21	3.61	3.16	2.53

STEEL CHANNELS.

15″ CHANNEL. SHAPE No. 31. 176¼ LBS. PER YARD.

Depth, 15″. Width of flange, $4\frac{5}{16}$″. Thickness of web, $\frac{13}{16}$″.

$$\text{Safe load in nett tons} = \frac{344.50}{\text{Span in feet}}.$$

Maximum shear = 39.74 tons.

Span limit for uniformly distributed load of twice the maximum shear = 4.33′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	57.42	0.04	353						
8	43.06	0.09	470						43.06
10	34.45	0.14	588				39.37	34.45	27.56
12	28.71	0.19	705		38.28	31.90	27.34	23.92	19.14
14	24.61	0.27	823	35.16	28.13	23.44	20.09	17.58	14.06
16	21.53	0.35	940	26.91	21.53	17.94	15.38	13.46	10.76
18	19.14	0.44	1058	21.27	17.02	14.18	12.15	10.63	8.51
20	17.23	0.56	1175	17.23	13.78	11.49	9.85	8.61	6.89
22	15.66	0.68	1293	14.24	11.39	9.49	8.14	7.12	5.70
24	14.35	0.81	1410	11.96	9.57	7.97	6.83	5.98	4.78
26	13.25	0.95	1528	10.19	8.15	6.79	5.82	5.09	4.08
28	12.30	1.09	1645	8.79	7.03	5.86	5.02	4.39	3.52
30	11.48	1.25	1763	7.65	6.12	5.10	4.37	3.82	3.06
32	10.77	1.43	1880	6.73	5.38	4.49	3.85	3.36	2.69
34	10.13	1.62	1998	5.96	4.77	3.97	3.41	2.98	2.38

STEEL CHANNELS.

15″ CHANNEL. SHAPE No. 31. 126¼ LBS. PER YARD.

Depth, 15″. Width of flange, $3\frac{63}{64}''$. Thickness of web, $\frac{31}{64}''$.

$$\text{Safe load in nett tons} = \frac{274.30}{\text{Span in feet}}.$$

Maximum shear = 17.67 tons.

Span limit for uniformly distributed load of twice the maximum shear = 7.76′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs per square foot.	250 lbs. per square foot.
6	45.72	0.04	254						
8	34.29	0.09	338					42.86	34.29
10	27.43	0.14	422		43.89	36.57	31.35	27.43	21.94
12	22.86	0.19	506	38.10	30.48	25.40	21.77	19.05	15.24
14	19.59	0.27	590	27.99	22.39	18.66	15.99	13.99	11.20
16	17.14	0.35	674	21.43	17.14	14.29	12.25	10.71	8.57
18	15.24	0.44	758	16.93	13.54	11.29	9.67	8.46	6.77
20	13.72	0.56	843	13.72	10.98	9.15	7.84	6.86	5.49
22	12.47	0.68	927	11.34	9.07	7.56	6.48	5.67	4.54
24	11.43	0.81	1010	9.53	7.62	6.35	5.45	4.76	3.81
26	10.55	0.95	1094	8.12	6.50	5.41	4.64	4.06	3.25
28	9.80	1.09	1178	7.00	5.60	4.67	4.00	3.50	2.80
30	9.14	1.25	1262	6.09	4.87	4.06	3.48	3.04	2.44
32	8.57	1.43	1347	5.36	4.29	3.57	3.06	2.68	2.14
34	8.07	1.62	1430	4.75	3.80	3.17	2.71	2.37	

STEEL CHANNELS.

12″ CHANNEL. SHAPE No. 32. 151½ LBS. PER YARD.

Depth, 12″. Width of flange, 3½″. Thickness of web, $\frac{15}{16}$″.

$$\text{Safe load in nett tons} = \frac{221.00}{\text{Span in feet}}.$$

Maximum shear = 40.61 tons.

Span limit for uniformly distributed load of twice the maximum shear = 2.72′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	36.83	0.06	303						
8	27.63	0.10	404				39.47	34.54	27.63
10	22.10	0.17	505	44.20	35.36	29.47	25.26	22.10	17.68
12	18.42	0.25	606	30.70	24.56	20.47	17.54	15.35	12.28
14	15.79	0.34	707	22.56	18.05	15.04	12.89	11.28	9.02
16	13.81	0.44	808	17.26	13.81	11.51	9.86	8.63	6.90
18	12.28	0.56	909	13.64	10.91	9.09	7.79	6.82	5.46
20	11.05	0.70	1010	11.05	8.84	7.37	6.31	5.52	4.42
22	10.05	0.85	1111	9.14	7.31	6.09	5.22	4.57	3.66
24	9.21	1.00	1212	7.68	6.14	5.12	4.39	3.84	3.07
26	8.50	1.17	1313	6.54	5.23	4.36	3.74	3.27	2.62
28	7.89	1.36	1414	5.64	4.51	3.76	3.22	2.82	2.26
30	7.37	1.56	1515	4.91	3.93	3.27	2.81	2.46	

STEEL CHANNELS.

12″ CHANNEL. SHAPE No. 32. 91 LBS. PER YARD.

Depth, 12″. Width of flange, 3″. Thickness of web, $\frac{7}{16}$″.

$$\text{Safe load in nett tons} = \frac{157.30}{\text{Span in feet}}.$$

Maximum shear = 13.97 tons.

Span limit for uniformly distributed load of twice the maximum shear = 5.63′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	26.22	0.06	181					43.70	34.96
8	19.66	0.10	242		39.32	32.77	28.09	24.57	19.66
10	15.73	0.17	303	31.46	25.17	20.97	17.98	15.73	12.58
12	13.11	0.25	363	21.85	17.48	14.57	12.49	10.92	8.74
14	11.24	0.34	424	16.06	12.85	10.71	9.18	8.03	6.42
16	9.83	0.44	484	12.29	9.83	8.19	7.02	6.14	4.92
18	8.74	0.56	545	9.71	7.77	6.47	5.55	4.85	3.88
20	7.87	0.70	606	7.87	6.30	5.25	4.50	3.93	3.15
22	7.15	0.85	666	6.50	5.20	4.33	3.71	3.25	2.60
24	6.55	1.00	727	5.46	4.37	3.64	3.12	2.73	2.18
26	6.05	1.17	788	4.65	3.72	3.10	2.66	2.32	1.86
28	5.62	1.36	848	4.01	3.21	2.67	2.29	2.00	
30	5.24	1.56	909	3.49	2.79	2.33	1.99	1.74	

STEEL CHANNELS.

12″ CHANNEL. SHAPE No. 34. 85½ LBS. PER YARD.

Depth, 12″. Width of flange, $2\frac{15}{16}$″. Thickness of web, ½″.

$$\text{Safe load in nett tons} = \frac{132.60}{\text{Span in feet}}.$$

Maximum shear = 17.34 tons.

Span limit for uniformly distributed load of twice the maximum shear = 3.82′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	22.10	0.07	171					36.83	29.47
8	16.58	0.10	228		33.16	27.63	23.69	20.72	16.58
10	13.26	0.17	285	26.52	21.22	17.68	15.15	13.26	10.61
12	11.05	0.25	342	18.42	14.74	12.28	10.53	9.21	7.37
14	9.47	0.34	399	13.53	10.82	9.02	7.73	6.77	5.41
16	8.29	0.44	456	10.36	8.29	6.91	5.92	5.18	4.15
18	7.37	0.56	512	8.19	6.55	5.46	4.68	4.09	3.28
20	6.63	0.70	569	6.63	5.30	4.42	3.79	3.32	2.65
22	6.03	0.85	626	5.48	4.38	3.65	3.13	2.74	2.19
24	5.53	1.00	683	4.61	3.69	3.07	2.63	2.31	1.84
26	5.10	1.17	740	3.92	3.14	2.61	2.24	1.96	1.57
28	4.74	1.37	797	3.39	2.71	2.26	1.94	1.65	1.36
30	4.42	1.56	854	2.95	2.36	1.97	1.69	1.47	1.18

STEEL CHANNELS.

12″ CHANNEL. SHAPE No. 34. 62⅔ LBS. PER YARD.

Depth, 12″. Width of flange, 2¾″. Thickness of web, $\frac{5}{16}$″.

$$\text{Safe load in nett tons} = \frac{109.20}{\text{Span in feet}}.$$

Maximum shear = 7.61 tons.

Span limit for uniformly distributed load of twice the maximum shear = 7.17′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	18.20	0.07	125				34.67	30.33	22.92
8	13.65	0.10	167	34.13	27.30	22.75	19.50	17.06	17.07
10	10.92	0.17	209	21.84	17.47	14.56	12.48	10.92	8.73
12	9.10	0.25	251	15.17	12.14	10.11	8.67	7.58	6.07
14	7.80	0.34	292	11.14	8.91	7.43	6.37	5.57	4.46
16	6.83	0.44	334	8.54	6.83	5.69	4.88	4.27	3.42
18	6.07	0.56	376	6.74	5.39	4.49	3.85	3.37	2.69
20	5.46	0.70	418	5.46	4.37	3.64	3.12	2.73	2.18
22	4.96	0.85	460	4.51	3.61	3.01	2.58	2.25	1.80
24	4.55	1.00	501	3.79	3.03	2.53	2.17	1.89	1.51
26	4.20	1.17	543	3.23	2.58	2.15	1.85	1.66	1.29
28	3.90	1.37	585	2.79	2.23	1.86	1.59	1.39	1.12
30	3.64	1.56	627	2.43	1.94	1.62	1.39	1.21	0.97

STEEL CHANNELS.

10″ CHANNEL. SHAPE No. 35. 129⅓ LBS. PER YARD.

Depth, 10″. Width of flange, 3½″. Thickness of web, $1\frac{1}{16}$″.

$$\text{Safe load in nett tons} = \frac{145.60}{\text{Span in feet}}.$$

Maximum shear = 40.25 tons.

Span limit for uniformly distributed load of twice the maximum shear = 1.81′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs per square foot.	250 lbs. per square foot.
6	24.27	0.05	260					40.45	32.36
8	18.20	0.11	347		36.40	30.33	26.00	27.75	18.20
10	14.56	0.19	434	29.12	23.30	19.41	16.64	14.56	11.65
12	12.13	0.28	520	20.22	16.18	13.48	11.55	10.11	8.09
14	10.40	0.39	607	14.86	11.89	9.91	8.49	7.43	5.95
16	9.10	0.52	694	11.38	9.10	7.59	6.50	5.69	4.55
18	8.09	0.65	781	8.99	7.19	5.99	5.14	4.49	3.55
20	7.28	0.80	867	7.28	5.82	4.85	4.16	3.64	2.91
22	6.62	0.98	954	5.65	4.52	3.77	3.23	2.83	2.26
24	6.07	1.19	1041	5.06	4.05	3.37	2.89	2.53	2.03
26	5.60	1.40	1128	4.31	3.45	2.87	2.46	2.16	1.73
28	5.20	1.61	1214	3.71	2.97	2.47	2.12	1.86	1.49
30	4.85	1.84	1301	3.23	2.58	2.15	1.85	1.62	1.29

STEEL CHANNELS.

10″ CHANNEL. SHAPE No. 35. 60⅔ LBS. PER YARD.

Depth, 10″. Width of flange, $2\frac{23}{32}$″. Thickness of web, ⅜″.

$$\text{Safe load in nett tons} = \frac{85.30}{\text{Span in feet}}.$$

Maximum shear = 10.12 tons.

Span limit for uniformly distributed load of twice the maximum shear = 4.21′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	14.21	0.05	121	47.37	37.90	31.58	27.07	23.69	18.95
8	10.66	0.11	162	26.65	21.32	17.77	15.23	13.32	10.66
10	8.53	0.19	202	17.06	13.65	11.37	9.75	8.53	6.83
12	7.11	0.28	243	11.85	9.48	7.90	6.77	5.93	4.74
14	6.09	0.39	283	8.70	6.96	5.80	4.97	4.35	3.48
16	5.33	0.52	324	6.66	5.33	4.44	3.81	3.33	2.67
18	4.74	0.65	364	5.27	4.22	3.51	3.01	2.64	2.11
20	4.26	0.80	404	4.26	3.41	2.84	2.44	2.13	1.71
22	3.88	0.98	445	3.53	2.82	2.35	2.02	1.77	1.41
24	3.55	1.19	485	2.96	2.37	1.97	1.69	1.48	1.19
26	3.28	1.40	526	2.52	2.02	1.68	1.44	1.26	1.01
28	3.05	1.61	566	2.18	1.74	1.45	1.25	1.09	
30	2.84	1.84	607	1.89	1.51	1.26	1.08		

STEEL CHANNELS.

10″ CHANNEL. SHAPE No. 36. 62⅔ LBS. PER YARD.

Depth, 10″. Width of flange, 2⅝″. Thickness of web, $\frac{7}{16}$″.

$$\text{Safe load in nett tons} = \frac{83.20}{\text{Span in feet}}.$$

Maximum shear = 13.05 tons.

Span limit for uniformly distributed load of twice the maximum shear = 3.18′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	13.87	0.05	125	46.23	36.98	30.82	26.42	23.11	18.49
8	10.40	0.11	167	26.00	20.80	17.33	14.86	13.00	10.40
10	8.32	0.19	209	16.64	13.31	11.09	9.51	8.32	6.66
12	6.93	0.28	251	11.55	9.24	7.70	6.60	5.77	4.62
14	5.94	0.39	292	8.49	6.79	5.66	4.85	4.24	3.40
16	5.20	0.52	334	6.50	5.20	4.33	3.71	3.25	2.60
18	4.62	0.65	376	5.13	4.10	3.42	2.93	2.56	2.05
20	4.16	0.80	418	4.16	3.33	2.77	2.38	2.08	
22	3.78	0.98	460	3.44	2.75	2.29	1.97		
24	3.47	1.19	501	2.89	2.31	1.93			
26	3.20	1.40	543	2.46	1.97				
28	2.97	1.61	585	2.12					
30	2.77	1.84	627	1.85					

STEEL CHANNELS.

10″ CHANNEL. SHAPE No. 36. 48½ LBS. PER YARD.

Depth, 10″. Width of flange, 2½″. Thickness of web, $\frac{5}{16}$″.

$$\text{Safe load in nett tons} = \frac{67.60}{\text{Span in feet}}.$$

Maximum shear = 7.46 tons.

Span limit for uniformly distributed load of twice the maximum shear = 4.52′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	11.27	0.05	97	37.57	30.06	25.05	21.47	18.78	15.03
8	8.45	0.11	129	21.13	16.90	14.09	12.07	10.57	8.45
10	6.76	0.19	162	13.52	10.82	9.01	7.73	6.76	5.41
12	5.63	0.28	194	9.38	7.50	6.25	5.36	4.69	3.75
14	4.83	0.39	226	6.90	5.52	4.60	3.94	3.45	2.76
16	4.23	0.52	259	5.29	4.23	3.53	3.02	2.64	2.12
18	3.76	0.65	291	4.18	3.34	2.79	2.39	2.09	1.67
20	3.38	0.80	323	3.38	2.70	2.25	1.93	1.69	1.35
22	3.07	0.98	356	2.79	2.23	1.86	1.59	1.39	1.12
24	2.82	1.19	388	2.35	1.88	1.56	1.34	1.18	0.94
26	2.60	1.40	420	2.00	1.60	1.33	1.14	1.00	
28	2.41	1.61	453	1.72	1.38	1.15	0.98		
30	2.25	1.84	485	1.50	1.20	1.00			

STEEL CHANNELS.

9″ CHANNEL. SHAPE No. 37. 52½ LBS. PER YARD.

Depth, 9″. Width of flange, 2½″. Thickness of web, $\frac{11}{32}$″.

$$\text{Safe load in nett tons} = \frac{68.90}{\text{Span in feet}}.$$

Maximum shear = 8.50 tons.

Span limit for uniformly distributed load of twice the maximum shear = 4.05′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	11.48	0.04	105	38.27	30.62	25.51	21.87	19.13	15.31
8	8.61	0.13	140	21.53	17.22	14.35	12.30	10.76	8.61
10	6.89	0.23	175	13.78	11.02	9.19	7.87	6.89	5.51
12	5.74	0.34	210	9.57	7.66	6.38	5.47	4.78	3.83
14	4.92	0.46	245	7.03	5.62	4.69	4.02	3.51	2.81
16	4.31	0.60	280	5.39	4.31	3.59	3.08	2.69	2.16
18	3.83	0.75	315	4.26	3.41	2.84	2.43	2.13	
20	3.45	0.92	350	3.45	2.76	2.30	1.97		
22	3.13	1.12	385	2.85	2.28	1.90			
24	2.87	1.34	420	2.39	1.91				
26	2.65	1.56	455	2.04					
28	2.46	1.82	490	1.76					
30	2.30	2.08	525	1.53					

STEEL CHANNELS.

9″ CHANNEL. SHAPE No. 38. 37½ LBS. PER YARD.

Depth, 9″. Width of flange, $2\frac{3}{16}$″. Thickness of web, ¼″.

$$\text{Safe load in nett tons} = \frac{48.10}{\text{Span in feet}}.$$

Maximum shear = 4.91 tons.

Span limit for uniformly distributed load of twice the maximum shear = 4.89′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	8.02	0.04	75	26.73	21.38	17.82	15.27	13.36	10.69
8	6.01	0.13	100	15.03	12.02	10.02	8.59	7.51	6.01
10	4.81	0.23	125	9.62	7.70	6.41	5.50	4.81	3.85
12	4.01	0.34	150	6.68	5.34	4.45	3.82	3.34	2.67
14	3.44	0.46	175	4.91	3.93	3.27	2.81	2.45	1.96
16	3.01	0.60	200	3.76	3.01	2.51	2.15	1.88	
18	2.67	0.75	225	2.97	2.38	1.98			
20	2.41	0.92	250	2.41	1.93				
22	2.19	1.12	275	1.99					
24	2.00	1.34	300						
26	1.85	1.56	325						
28	1.72	1.82	350						
30	1.60	2.08	375						

Span limit for tabular safe load = 5.40′.

STEEL CHANNELS.

8″ CHANNEL. SHAPE No. 39. 40½ LBS. PER YARD.

Depth, 8″. Width of flange, $2\frac{5}{16}$″. Thickness of web, $\frac{5}{16}$″.

$$\text{Safe load in nett tons} = \frac{45.80}{\text{Span in feet}}.$$

Maximum shear = 6.97 tons.

Span limit for uniformly distributed load of twice the maximum shear = 3.29′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	7.64	0.07	81	25.47	20.38	16.98	14.55	12.74	10.19
8	5.73	0.14	108	14.33	11.46	9.55	8.19	7.17	5.73
10	4.58	0.26	135	9.16	7.33	6.11	5.23	4.58	3.67
12	3.82	0.39	162	6.37	5.10	4.25	3.64	3.19	2.55
14	3.27	0.52	189	4.67	3.74	3.11	2.67	2.34	1.87
16	2.86	0.65	216	3.58	2.86	2.39	2.05	1.79	1.43
18	2.55	0.86	243	2.83	2.26	1.89	1.62	1.42	1.13
20	2.29	1.04	270	2.29	1.83	1.53	1.31	1.15	0.92

STEEL CHANNELS.

8″ CHANNEL. SHAPE No. 40. 30⅓ LBS. PER YARD.

Depth, 8″. Width of flange, $2\frac{1}{16}$″. Thickness of web, ¼″.

$$\text{Safe load in nett tons} = \frac{33.20}{\text{Span in feet}}.$$

Maximum shear = 4.78 tons.

Span limit for uniformly distributed load of twice the maximum shear = 3.49′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	5.53	0.07	61	18.43	14.74	12.29	10.53	9.22	7.37
8	4.14	0.14	81	10.35	8.28	6.90	5.91	5.18	4.14
10	3.31	0.26	101	6.62	5.30	4.41	3.78	3.31	2.65
12	2.76	0.39	121	4.60	3.68	3.07	2.63	2.30	1.84
14	2.37	0.52	142	3.39	2.71	2.26	1.94	1.69	1.36
16	2.07	0.65	162	2.59	2.07	1.73	1.48	1.30	1.04
18	1.84	0.86	182	2.04	1.63	1.36	1.17	1.02	
20	1.66	1.04	202	1.66	1.33	1.11	0.95		

Span limit for tabular safe load = 5.10′.

STEEL CHANNELS.

7″ CHANNEL. SHAPE No. 41. 35½ LBS. PER YARD.

Depth, 7″. Width of flange, 2¼″. Thickness of web, $\frac{5}{16}$″.

$$\text{Safe load in nett tons} = \frac{35.10}{\text{Span in feet}}.$$

Maximum shear = 6.53 tons.

Span limit for uniformly distributed load of twice the maximum shear = 2.69′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	5.85	0.06	71	19.50	15.60	13.00	11.14	9.75	7.80
8	4.39	0.17	95	10.98	8.78	7.32	6.27	5.49	4.39
10	3.51	0.30	118	7.02	5.61	4.68	4.01	3.51	2.81
12	2.93	0.44	142	4.88	3.90	3.25	2.79	2.44	1.95
14	2.51	0.64	166	3.59	2.87	2.39	2.05	1.79	
16	2.19	0.78	189	2.74	2.19	1.83			
18	1.95	0.99	213	2.17	1.74				
20	1.76	1.22	237	1.76		Span limit for tabular safe load = 5.70′.			

STEEL CHANNELS.

7″ CHANNEL. SHAPE No. 42. 25¼ LBS. PER YARD.

Depth, 7″. Width of flange, 2″. Thickness of web, $\frac{7}{32}$″.

$$\text{Safe load in nett tons} = \frac{26.00}{\text{Span in feet}}.$$

Maximum shear = 3.66 tons.

Span limit for uniformly distributed load of twice the maximum shear = 3.55′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	4.33	0.06	51	14.43	11.54	9.62	8.25	7.21	5.77
8	3.25	0.17	67	8.13	6.50	5.42	4.65	4.06	3.25
10	2.60	0.30	84	5.20	4.16	3.47	2.97	2.60	2.08
12	2.17	0.44	101	3.62	2.90	2.41	2.07	1.81	1.45
14	1.86	0.64	118	2.66	2.13	1.77	1.52	1.33	
16	1.63	0.78	134	2.04	1.63	1.36	1.17		
18	1.44	0.99	152	1.60	1.28	Span limit for tabular safe load = 5.10′.			
20	1.30	1.22	168	1.30					

STEEL CHANNELS.

6″ CHANNEL. SHAPE No. 43. 30⅓ LBS. PER YARD.

Depth, 6″. Width of flange, 2″. Thickness of web, ¼″.

$$\text{Safe load in nett tons} = \frac{28.20}{\text{Span in feet}}.$$

Maximum shear = 4.40 tons.

Span limit for uniformly distributed load of twice the maximum shear = 3.20′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	4.70	0.07	61	15.67	12.54	10.45	8.95	7.84	6.27
8	3.52	0.20	81	8.80	7.04	5.87	5.03	4.40	3.52
10	2.81	0.34	101	5.62	4.50	3.75	3.21	2.81	2.25
12	2.35	0.49	121	3.92	3.14	2.61	2.24	1.96	1.57
14	2.01	0.75	142	2.87	2.30	1.91	1.64	1.44	1.65
16	1.76	0.91	162	2.20	1.76	1.47	1.26	1.10	
18	1.57	1.13	182	1.74	1.39	1.16	0.99		
20	1.41	1.40	202	1.41	1.13	0.94	Span limit for tabular safe load = 5.10′.		

STEEL CHANNELS.

6″ CHANNEL. SHAPE No. 44. 22¾ LBS. PER YARD.

Depth, 6″. Width of flange, $1\frac{11}{16}$″. Thickness of web, $\frac{3}{16}$″.

$$\text{Safe load in nett tons} = \frac{20.80}{\text{Span in feet}}.$$

Maximum shear = 2.66 tons.

Span limit for uniformly distributed load of twice the maximum shear = 3.91′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	3.47	0.07	45	11.57	9.26	7.71	6.61	5.78	4.63
8	2.60	0.20	60	6.50	5.20	4.33	3.71	3.25	2.60
10	2.08	0.34	76	4.16	3.33	2.77	2.38	2.08	1.66
12	1.73	0.49	91	2.88	2.30	1.92	1.65	1.44	1.15
14	1.49	0.75	106	2.13	1.70	1.42	1.22	1.06	
16	1.30	0.91	121	1.63	1.30	1.09			
18	1.16	1.13	136	1.29	1.03	Span limit for tabular safe load = 4.20′.			
20	1.04	1.40	152	1.04					

STEEL CHANNELS.

5″ CHANNEL. SHAPE No. 45. 26¼ LBS. PER YARD.

Depth, 5″. Width of flange, 1⅞″. Thickness of web, ¼″.

$$\text{Safe load in nett tons} = \frac{19.50}{\text{Span in feet}}.$$

Maximum shear = 3.96 tons.

Span limit for uniformly distributed load of twice the maximum shear = 2.46′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of					
				100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	3.25	0.14	52	10.83	8.66	7.22	6.19	5.41	4.33
8	2.44	0.27	70	6.10	4.88	4.07	3.49	3.05	2.44
10	1.95	0.43	88	3.90	3.12	2.60	2.23	1.95	1.56
12	1.63	0.62	104	2.72	2.18	1.81	1.55	1.36	1.09
14	1.39	0.78	123	1.99	1.59	1.33	1.14		
16	1.22	1.04	140	1.53	1.22	1.02			
18	1.08	1.30	158	1.20	0.96	Span limit for tabular safe load = 4.80′.			
20	0.98	1.69	176	0.98					

STEEL CHANNELS.

5″ CHANNEL. SHAPE No. 46. 17¼ LBS. PER YARD.

Depth, 5″. Width of flange, 1⅜″. Thickness of web, $\frac{3}{16}$″.

$$\text{Safe load in nett tons} = \frac{13.00}{\text{Span in feet}}.$$

Maximum shear = 2.54 tons.

Span limit for uniformly distributed load of twice the maximum shear = 2.56′.

Span, in feet.	Safe load, in nett tons.	Deflexion, in inches.	Weight of beam.	Distance apart, in feet, centre to centre of beams, for safe loads of 100 lbs. per square foot.	125 lbs. per square foot.	150 lbs. per square foot.	175 lbs. per square foot.	200 lbs. per square foot.	250 lbs. per square foot.
6	2.17	0.14	34	7.23	5.78	4.82	4.13	3.61	2.89
8	1.62	0.27	46	4.05	3.24	2.70	2.31	2.02	1.62
10	1.30	0.43	59	2.60	2.08	1.73	1.49	1.30	1.04
12	1.08	0.62	71	1.80	1.44	1.20	1.03	0.90	0.72
14	0.93	0.78	83	1.33	1.06	0.89	0.76	0.67	
16	0.81	1.04	95	1.01	0.81	0.67	0.58		
18	0.72	1.30	107	0.80	0.64	0.53			
20	0.65	1.69	120	0.65	0.52		Span limit for tabular safe load = 4.40′.		

ON DETERMINING THE CAPACITY, ETC., OF BEAMS AND CHANNELS.

Let S = area of section.

l = length of span.

w = load per linear unit of beam.

W = total load uniformly distributed.

M_o = maximum bending moment of external forces.

h = height of shape.

y = distance from neutral axis to that edge of shape which first ruptures, and which in symmetrical sections is one-half the height.

f = extreme fibre stress (generally taken in tons per square inch) on that side of the neutral axis which first ruptures.

I = maximum moment of inertia of section.

J = minimum moment of inertia of section.

r_I = maximum radius of gyration, $\sqrt{\frac{I}{S}}$

r_J = minimum radius of gyration, $\sqrt{\frac{J}{S}}$

C = coefficient for one foot span.

For iron shapes $= \frac{8\,I}{h} = 4\,R.$

For steel shapes $= \frac{10.4\,I}{h} = 5.2\,R.$

R = modulus of section $= \frac{I}{y} =$ for a symmetrical shape $= \frac{2\,I}{h}.$

$\triangle$ = maximum deflexion (generally given in inches).

Let F_o = the maximum shear permissible.

For iron shapes, $$F_o = \frac{3.00 \text{ tons}}{1 + \frac{\left[\frac{h\sqrt{2}}{t}\right]^2}{3000}}$$

For steel shapes, $$F_o = \frac{4.00 \text{ tons}}{1 + \frac{\left[\frac{h\sqrt{2}}{t}\right]^2}{3000}}$$

q = a factor dependent upon form of section, and is the ratio

$$q = \frac{I}{m' h^2 S} = \frac{I}{h\,y\,s} = \frac{r^2}{h\,y}$$

since $\frac{I}{S}$ = square of radius of gyration.

If the shape is symmetrical, $y = \frac{h}{2}$, or $m' = \frac{1}{2}$; then

$$q = \frac{2\,I}{h^2\,S} = 2\,\frac{R^2}{h^2} = 2\left[\frac{R}{h}\right]^2$$

In the above, I denotes simply *a* moment of inertia. If the least moment of inertia be in question, the above relations are also applicable, replacing I by J, and r by r_J, and h and y being taken in the direction of the least moment of inertia.

E = coefficient of elasticity, which for

iron shapes = 13,000 tons per square inch.
steel shapes = 14,500 tons per square inch.

$\triangle = \frac{5}{384}\,\frac{W\,l^3}{E\,I}$ for beam supported at both ends, and uniformly loaded over its entire length.

Let $\triangle = \frac{W\,l^3}{8\,E\,I}$ for beam fixed at one end, and uniformly loaded over its entire length.

$\triangle = \frac{P\,l^3}{48\,E\,I}$ for beam supported at both ends, and having a concentrated load, P, at the centre.

$\triangle = \frac{P\,l^3}{3\,E\,I}$ for beam fixed at one end, and loaded at the other.

$\triangle = \frac{P\,l^3}{192\,E\,I}$ for beam fixed at both ends, and having a concentrated load, P, at the centre.

$\triangle = \frac{W\,l^3}{307\,E\,I}$ for beam fixed at both ends, and uniformly loaded over its entire length.

The relation between the external and molecular forces of a beam subjected to transverse loading is expressed by

$$M_o = \frac{f\,I}{y} \qquad (1)$$

the second member of which is called the *moment of resistance.*

When the beam is supported at its ends, and uniformly loaded over its entire length, the maximum moment due to external forces is at the centre of the beam, and is given by the expression, $M_o = \frac{W\,l}{8}$. The moment of resistance of the beam should at least equal this, and for beams of symmetrical sections, in which y is equal to one-half the height, the general expression (1) becomes

$$\frac{W\,l}{8} = \frac{2\,f\,I}{h} \qquad (2)$$

from which we get

$$W = \frac{16\,f\,I}{l\,h} \qquad (3)$$

If, as is usually the case, we take the length of beam in feet and the height in inches, then equation (3) becomes

$$W = \frac{4\,f\,I}{3\,l'\,h''} \qquad (4)$$

in which l' denotes the span of beam in feet, and h'' the height in inches.

In beams of iron we take as the safe working extreme fibre stress f, 6.0 tons per square inch, this being two-sevenths ($\frac{2}{7}$) of the modulus of rupture.

In beams of steel we take as the safe working extreme fibre stress f, 7.8 tons per square inch, which is likewise two-sevenths ($\frac{2}{7}$) of the modulus of rupture of steel beams.

Then, for iron beams, we get from (4), by making $f = 6.0$ tons,

$$W = \frac{8\,I}{l'\,h''}. \qquad (5)$$

and for steel beams we get, by making $f = 7.8$ tons,

$$W = \frac{10.4\,I}{l'\,h''} \qquad (6)$$

in both of which expressions W is the safe load, in nett tons, uniformly distributed.

If we consider the span l' to be *one foot*, then we have what has been called the coefficient for one foot of span,—*i.e.*,

For iron beams, $$C = \frac{8\,I}{h''} \qquad (7)$$

For steel beams, $$C = \frac{10.4\,I}{h''} \qquad (8)$$

Now, on page 148, we have called $\frac{I}{y}$ the *modulus* of the section, and denoted it by the letter R.

As in symmetrical sections $y = \frac{1}{2}\,h''$, the *modulus* for such sections is

$$R = \frac{2\,I}{h''} \qquad (9)$$

Whence the safe load could be written,

For iron beams, $$W = \frac{4\,R}{l'} \qquad (10)$$

For steel beams, $$W = \frac{5.2\ R}{l'} \qquad (11)$$

and the coefficients for one foot span could be written,

For iron beams, $$C = 4\ R \qquad (12)$$

For steel beams, $$C = 5.2\ R \qquad (13)$$

From the foregoing expressions many useful relations can be obtained.

I. Given the load in nett tons, W, on a beam; l, the span in feet; h, the height in inches; I, the moment of inertia of the beam. Required the extreme fibre stress f?

$$f = \frac{3}{4}\ W\ l'\ \frac{h''}{I} \qquad (14)$$

II. Given the load in nett tons, W, on a beam; l′, the span in feet; f, the extreme fibre stress. Required the *modulus of the section?*

$$R = \frac{2\ I}{h''} = \frac{3}{2}\ \frac{W\,l'}{f} \qquad (15)$$

III. Given the load in nett tons, W, on a beam; f, the extreme fibre stress; h″, the height of the beam, and I its moment of inertia; or R, the modulus of the section. Required the span for which the beam will safely carry the assumed load, W?

$$l = \frac{4}{3}\ \frac{f}{W}\ \frac{I}{h''} = \frac{2}{3}\ \frac{f}{W}\ \frac{2\ I}{h''} = \frac{2\,f}{3W}\ R \qquad (16)$$

IV. Given the span l′ in feet; the extreme fibre stress, f; the height, h″ of the beam, and I, its moment of inertia; or R, the modulus of the section. Required the load which the beam will carry?

$$W = \frac{4}{3}\ \frac{f\ I}{l'\ h''} = \frac{2}{3}\ \frac{f}{l'}\ \frac{2\ I}{h''} = \frac{2}{3}\ \frac{f}{l'}\ R \qquad (17)$$

Examples on the use of the foregoing expressions:

EXAMPLE I. Given a 12″ **I** beam of iron, 125 pounds per yard, whose span centre to centre of end bearings is 10 feet, carrying a load of 15 tons, uniformly distributed over its length. Required the extreme fibre stress, f?

Here

$$W = 15.0 \text{ tons};\ l' = 10.0 \text{ feet};\ h = 12''$$

and referring to the table "On the Properties of **I** Beams," page 159, we find for a 12″ **I** beam, 125 pounds per yard, the moment of inertia I to be 279.

Making these substitutions in expression (14), we get

$$f = \frac{3}{4} \times 15.0 \times 10 \times \frac{12}{279} = 4.84 \text{ tons per square inch.}$$

EXAMPLE II. Given a load of 9.75 tons, uniformly distributed on a span whose length centre to centre of end bearings is 12.0 feet, and having a height limiting us to the use of a $10\frac{1}{2}''$ **I** beam. Required the moment of inertia of the necessary $10\frac{1}{2}''$ **I** beam, assuming the extreme fibre stress to be 6.0 tons?

Here we have

$$W = 9.75 \text{ tons};\ l' = 12.0 \text{ feet};\ h = 10\tfrac{1}{2}'';\ f = 6.0$$

Making these substitutions in expression (15), we get

$$R = \frac{2\ I}{10\frac{1}{2}''} = \frac{3}{2} \times \frac{9.75 \times 12.0}{6.0}$$

$$= \frac{I}{5.25} = 29.25$$

i.e.,

$$R = 29.25$$
$$I = 29.25 \times 5.25 = 153.56$$

Referring to the table "On the Properties of **I** Beams," we find that a $10\frac{1}{2}''$ **I** beam of *iron*, 90 pounds per yard,

shape No. 10, has a value of R = 29.0, and a moment of inertia = 151. Hence this shape meets the requirements.

EXAMPLE III. Given a 12″ I beam of iron, 125 pounds per yard, whose moment of inertia is, as per table "On the Properties of I Beams," 279.0; or whose modulus R is 46.25; also, given the load to be carried is 9.25 tons, and the extreme fibre stress to be limited to 6.0 tons. Required the span centre to centre of end bearings, for which this beam could be used?

We have, then,

$$h'' = 12''; \; I = 279.0; \; R = 46.25; \; W = 9.25 \text{ tons}; \; f = 6.0 \text{ tons per square inch.}$$

Substituting these values in expression (16), we get

$$l' = \frac{4}{3} \times \frac{6.0}{9.25} \times \frac{279.0}{12} = 20.00 \text{ feet};$$

or, using the modulus R instead of the moment of inertia I, we get from (16)

$$l' = \frac{2}{3} \times \frac{6.0}{9.25} \times 46.25 = 20.00 \text{ feet}.$$

Thus, 20.0 feet is the limiting span of this beam, for the assumed load and fibre stress.

EXAMPLE IV. Suppose we have a span of 15 feet, and that we wish to use a 15″ I beam of wrought iron, 150 pounds per yard. Required the safe load which we can put on this beam, when the fibre stress is limited to 5.0 tons per square inch?

We have given, in the table "On the Properties of I Beams," R = 70.50. We also have given l′ = 15.0, and f = 5.0 tons. Inserting these values in expression (17), we get

$$W = \frac{2}{3} \times \frac{5.0}{15.0} \times 70.5 = 15.66 \text{ tons};$$

that is, our safe load is 15.66 tons, uniformly distributed over length of beam.

ON THE

PROPERTIES OF I BEAMS AND CHANNELS

OF IRON AND STEEL,

MANUFACTURED
BY THE

POTTSVILLE IRON AND STEEL CO.

The tables "On the Properties of I Beams and Channels" are calculated for the minimum and maximum weight to which these shapes are rolled.

The plates illustrate how the increase of weight is effected, which is simply by increasing the distance apart of the rolls; consequently, the increase in width in flanges is the same as increase in thickness of web.

I beams, channels, and angle irons may be rolled to any weight intermediate between the minimum and maximum weights as given. T iron can be rolled to but one weight.

Columns Nos. 10 and 11 in the tables for I beams and channels, pages 159, 160, give coefficients, by means of which the safe uniformly distributed load for any I beam or channel on the list can at once be obtained, when we know the span.

We have only to divide the coefficient by the span in feet, when the result is the safe load in nett tons, uniformly distributed, that the I beam or channel will carry.

The fibre stresses upon which these coefficients are based are for iron shapes, 6.0 tons per square inch; for steel shapes, 7.8 tons per square inch.

Should any case arise in which a lower fibre stress is desirable, the coefficient is simply reduced in the same proportion. For example: the coefficient for a fibre stress of 6.0 tons per square inch on a 12″ I beam of iron, 125 pounds per yard, is given by the table as 185. Should we wish the fibre stress to be but 4.0 tons, this being two-thirds of 6.0 tons, the coefficient is reduced in same proportion,—viz., to

$\frac{2}{3} \times 185 = 123.33.$

The resistance to bending of a beam of any kind is proportional to the *modulus of the section* of the beam.

If two beams of different forms be subjected to the same loading, that one will be the more economical which, with a given value of the *modulus of section*, has the smaller sectional area, S. In other words, the greater the ratio $\frac{R}{S}$, the more economical the beam. For example: looking in the tables on pages 159, 160, we find that a 6″ **I** beam of 5.0 square inches sectional area has a modulus of 9.00, and also that an 8″ channel of 4.00 square inches sectional area has a modulus of 9.00. Thus it is seen that, for the same modulus in each case, the 8″ channel has 20 per cent. less sectional area than the 6″ **I** beam, and hence weighs 20 per cent. less for a given length; whence the 8″ channel is the more economical shape. Moreover, it is a stiffer shape than the 6″ **I** beam, for, with the same loads and span, that shape has the less deflexion, because its moment of inertia is greater. Thus, for the 6″ **I** beam of 5.0 square inches area, the value of I is 27.0; whilst that for the 8″ channel of 4.00 square inches sectional area is 35.25. Hence, if these shapes be protected against *lateral* deflexion, it would be more economical to use the 8″ channel than the 6″ **I** beam, for the weakness of the channel is in its small width of flange, it having only a flange width of $2\frac{5}{16}$″, whilst the **I** beam has $3\frac{9}{32}$″.

In columns 8 of the tables on pages 159, 160, we have given, for each shape, the values of what Rankine has called q, which is the ratio

$$\frac{2\,I}{h^2\,S}$$

that is,

$$q = \frac{2\,I}{h^2\,S} = \frac{R}{h\,S} \qquad (18)$$

This shows that for two beams of the same depth, *that one is the more economical which has the greater value of the ratio* $\frac{R}{S}$, *or, in other words, that whose value of* q *is the greater.*

For example: consider shape No. 34 in the list of channels,—viz., the 12″ channel of 6.20 square inches sectional area, and the 12″ channel of 8.45 square inches sectional area. The former has $q = 0.281$ and $R = 21.0$; the latter has $q = 0.251$ and $R = 25.50$. Again, the former has $\frac{R}{S} = \frac{21}{6.2} = 3.387$; and the latter has $\frac{R}{S} = \frac{25.50}{8.45} = 3.002$. It is evident, then, that the 12″ channel, 6.2 square inches area, has a greater capacity for its weight than the 12″ channel, 8.45 square inches area. Thus it appears that the strength of beams does not increase in proportion to their increase of weight. We should then, always use the minimum or standard section of a shape, rather than one obtained by widening the rolls. Of course, this applies only to shapes subjected to transverse loading. From the values of q given in the tables on pages 159, 160, we can then at once see the relative economy of the shapes.

Another very desirable use to which these values of q can be put is as follows:

From the fundamental expression

$$M_o = f \frac{I}{y} \qquad \text{see (1), page 150}$$

which, for symmetrical shapes, becomes

$$M_o = f \cdot \frac{2I}{h}$$

we get, by substituting for $\frac{2I}{h}$, its equivalent, qh S,

$$M_o = f\, qh\, S \tag{19}$$

Whence, transposing,

$$S = \frac{M_o}{f\, qh} = \frac{M_o}{f} \cdot \frac{1}{qh} \tag{20}$$

i.e., area of shape for given values of $\frac{M_o}{f}$ is inversely proportional to qh; that is to say, the greater the value of qh, the less the area of beam required to resist the bending moment M_o, with an extreme fibre stress, f. For example:

suppose we have given a load of 13 tons uniformly distributed over a span of 14.0′; then $M_o = \frac{13 \times 14' \times 12''}{8} =$ 273 inch-tons bending moment at centre. The extreme fibre stress is to be limited to 6.0 tons; then $\frac{M_o}{f} = \frac{273}{6} =$ 45.5; whence

$$S = \text{area of beam required} = \frac{M_o}{f} \cdot \frac{1}{qh} = \frac{45.5}{qh}$$

Now, looking at the table "On Properties of **I** Beams," we find for a 12″ **I** beam, 12.5 square inches, $q = 0.310$, whence $qh = 0.31 \times 12'' = 3.72$; and for a $10\frac{1}{2}''$ **I** beam, 13.5 square inches, $q = 0.316$, whence $qh = 10\frac{1}{2} \times 0.316 = 3.32$; whence for the former,

$$S = \frac{45.5}{3.72} = 12.20 \text{ square inches,}$$

and for the latter,

$$S = \frac{45.5}{3.32} = 13.70 \text{ square inches;}$$

that is, using a 12″ **I** beam, we need only 12.2 square inches of area; whilst, if we use a $10\frac{1}{2}''$ **I** beam, we require 13.70 square inches of area. It is evident, then, that for *the same maximum moment, and same extreme fibre stresses, that beam is the more economical which has the larger value of* qh.

By inspection of the tables on pages 159, 160, we see that for the standard or minimum roll of **I** beams, the value of q departs but little from 0.31. For channels, the value of q for the standard rolls is about 0.28, and for the heavier rolls q is about 0.25. Thus,

I beams, standard rolls, $q = 0.31$.
Channels, minimum rolls, $q = 0.28$.
Channels, maximum rolls, $q = 0.25$.

Now, substituting these constants in equation (19), we get

$M_o = 0.31\ f\,h\,S$, for **I** beams of standard rolls.
$M_o = 0.28\ f\,h\,S$, for channels of minimum rolls.
$M_o = 0.25\ f\,h\,S$, for channels of maximum rolls.

PROPERTIES OF

I BEAMS OF IRON AND STEEL.

1	2	3	4	5	6	7	8	9	10	11	12	13
					Neutral axis at centre of shape and perpendicular to web.						Neutral axis coincident with web.	
									Coefficient for one foot span.			
Shape No.	Depth of shape, in inches.	Area of shape, in square inches.	Width of flange, in inches.	Thickness of web, in inches.	Maximum moment of inertia I.	Radius of gyration r_r.	$q = \frac{2\,I}{h^2\,S}$.	$R = \frac{2\,I}{h}$.	Iron I beams $C = \frac{8\,I}{h}$.	Steel I beams $C = \frac{10.4\,I}{h}$.	Minimum moment of inertia J.	Radius of gyration r_J.
1	15	25.0	$5\frac{7}{8}$	$\frac{7}{8}$	813.0	6.38	0.289	108.0	432.0	563.7	40.84	1.28
2	15	20.0	$5\frac{9}{16}$	$\frac{5}{8}$	694.0	5.89	0.309	92.5	370.0	481.0	33.79	1.30
3	15	15.0	5	$\frac{15}{32}$	528.0	5.93	0.313	70.5	282.0	366.6	18.34	1.10
4	15	12.5	$4\frac{7}{8}$	$\frac{7}{16}$	430.0	5.87	0.306	57.0	228.0	296.4	13.13	1.03
5	12	17.0	$5\frac{3}{8}$	$\frac{11}{16}$	367.0	4.65	0.300	61.0	244.0	317.2	24.47	1.20
6	12	12.5	$4\frac{7}{8}$	$\frac{1}{2}$	279.0	4.72	0.310	46.25	185.0	240.9	14.50	1.08
7	12	10.0	$4\frac{7}{16}$	$\frac{7}{16}$	218.0	4.66	0.303	36.0	144.0	187.2	8.74	0.94
8	$10\frac{1}{2}$	13.5	5	$\frac{17}{32}$	239.0	4.17	0.316	45.5	182.0	236.7	17.90	1.15
9	$10\frac{1}{2}$	10.5	$4\frac{3}{8}$	$\frac{1}{2}$	176.0	4.08	0.301	33.5	134.0	174.3	9.52	0.95
10	$10\frac{1}{2}$	9.0	$4\frac{1}{8}$	$\frac{13}{32}$	151.0	4.12	0.309	29.0	116.0	149.6	7.36	0.90
11	10	10.5	$4\frac{5}{8}$	$\frac{1}{2}$	161.0	3.92	0.307	32.25	129.0	167.7	11.08	1.03
12	10	9.0	$4\frac{3}{8}$	$\frac{7}{16}$	139.0	3.93	0.310	28.0	111.0	145.6	8.30	0.96
13	9	9.0	$4\frac{3}{8}$	$\frac{1}{2}$	110.0	3.50	0.302	24.5	98.0	127.4	8.18	0.95
14	9	8.5	$4\frac{1}{4}$	$\frac{7}{16}$	107.0	3.54	0.309	24.0	96.0	124.2	7.60	0.94
15	9	7.0	4	$\frac{3}{8}$	83.0	3.45	0.294	18.5	74.0	96.2	5.37	0.88
16	8	8.0	$4\frac{5}{32}$	$\frac{1}{2}$	77.0	3.10	0.300	19.25	77.0	100.1	6.60	0.91
17	8	6.5	4	$\frac{5}{16}$	69.0	3.26	0.332	17.0	68.0	88.4	5.83	0.95
18	7	6.5	$3\frac{9}{16}$	$\frac{29}{64}$	48.0	2.72	0.300	13.75	55.0	71.5	4.11	0.79
19	7	5.5	$3\frac{7}{16}$	$\frac{21}{64}$	43.0	2.80	0.320	12.5	50.0	65.4	3.51	0.80
20	6	5.0	$3\frac{9}{32}$	$\frac{13}{32}$	27.0	2.33	0.301	9.0	36.0	46.8	2.65	0.73
21	6	4.0	$3\frac{1}{8}$	$\frac{1}{4}$	24.0	2.44	0.332	8.0	32.0	41.6	2.22	0.74
22	5	4.0	$2\frac{15}{16}$	$\frac{3}{8}$	16.0	1.94	0.301	6.25	25.0	33.3	1.75	0.66
23	5	3.0	$2\frac{3}{4}$	$\frac{3}{16}$	12.0	2.00	0.320	4.8	19.2	25.0	1.39	0.68
24	4	3.0	$2\frac{7}{16}$	$\frac{27}{64}$	7.0	1.50	0.281	3.5	14.0	18.2	0.82	0.52
25	4	2.4	$2\frac{1}{4}$	$\frac{5}{16}$	5.6	1.53	0.293	2.8	11.4	14.56	0.58	0.51
26	4	1.8	$2\frac{1}{8}$	$\frac{3}{16}$	4.4	1.56	0.306	2.2	8.8	11.4	0.40	0.47

PROPERTIES OF

CHANNEL BEAMS OF IRON AND STEEL.

1	2	3	4	5	6	7	8	9	10	11	12	13	14
					Neutral axis at centre of shape and perpendicular to web.						Neutral axis parallel to web.		
									Coefficient for one foot span.				
Shape No.	Depth of shape, in inches.	Area of shape, in square inches.	Width of flange, in inches.	Thickness of web, in inches.	Maximum moment of inertia I.	Radius of gyration r_I.	$q = \frac{2I}{h^2 S}$.	$R = \frac{2I}{h}$.	Iron channel $C = \frac{8I}{h}$.	Steel channel $C = \frac{10.4I}{h}$.	Minimum moment of inertia J.	Radius of gyration r_J.	Distance of neutral axis from outside of web.
30	15	22.5	$5\frac{5}{64}$	$1\frac{5}{64}$	623.0	5.26	0.246	83.0	332.0	431.6	39.32	1.32	1.25
30	15	17.5	$4\frac{3}{4}$	$\frac{3}{4}$	527.0	5.48	0.268	70.25	281.0	365.3	31.41	1.34	1.23
31	15	17.45	$4\frac{5}{16}$	$\frac{13}{16}$	497.0	5.34	0.254	66.25	265.0	344.5	23.13	1.15	1.03
31	15	12.5	$3\frac{63}{64}$	$\frac{31}{64}$	396.0	5.63	0.282	52.75	211.0	274.3	17.54	1.18	1.08
32	12	15.0	$3\frac{1}{2}$	$\frac{15}{16}$	255.0	4.13	0.237	42.5	170.0	221.0	11.43	0.87	1.01
32	12	9.0	3	$\frac{7}{16}$	181.5	4.49	0.280	30.25	121.0	157.3	7.11	0.89	0.84
33	12	8.65	$2\frac{13}{16}$	$\frac{1}{2}$	159.0	4.28	0.255	26.5	106.0	137.8	4.98	0.76	0.68
33	12	6.4	$2\frac{5}{8}$	$\frac{5}{16}$	133.0	4.56	0.288	22.0	88.0	114.4	3.92	0.78	0.70
34	12	8.45	$2\frac{15}{16}$	$\frac{1}{2}$	153.0	4.25	0.251	25.5	102.0	132.6	5.04	0.77	0.68
34	12	6.2	$2\frac{3}{4}$	$\frac{5}{16}$	125.5	4.50	0.281	21.0	84.0	109.2	4.00	0.80	0.71
35	10	12.875	$3\frac{1}{2}$	$1\frac{1}{16}$	140.0	3.29	0.217	28.0	112.0	145.6	7.79	0.78	0.84
35	10	6.0	$2\frac{23}{32}$	$\frac{3}{8}$	82.0	3.69	0.272	16.5	66.0	85.3	3.73	0.79	0.69
36	10	6.2	$2\frac{5}{8}$	$\frac{7}{16}$	80.0	3.59	0.258	16.0	64.0	83.2	3.02	0.70	0.61
36	10	4.8	$2\frac{1}{2}$	$\frac{5}{16}$	65.0	3.69	0.272	13.0	52.0	67.6	2.40	0.71	0.59
37	9	8.65	3	$\frac{23}{32}$	83.0	3.10	0.237	18.5	74.0	96.2	4.90	0.75	0.74
37	9	5.2	$2\frac{1}{2}$	$\frac{11}{32}$	60.0	3.39	0.284	13.25	53.0	68.9	2.81	0.74	0.68
38	9	5.42	$2\frac{3}{8}$	$\frac{7}{16}$	53.5	3.14	0.244	12.0	48.0	62.4	2.04	0.61	0.54
38	9	3.7	$2\frac{3}{16}$	$\frac{1}{4}$	42.0	3.38	0.282	9.25	37.0	48.1	1.52	0.64	0.55
39	8	7.0	$2\frac{11}{16}$	$\frac{11}{16}$	51.0	2.70	0.228	12.75	51.0	66.3	2.85	0.64	0.63
39	8	4.0	$2\frac{5}{16}$	$\frac{5}{16}$	35.25	2.97	0.260	9.0	36.0	45.8	1.78	0.67	0.59
40	8	3.5	$2\frac{1}{8}$	$\frac{5}{16}$	28.25	2.84	0.253	7.0	28.0	36.7	1.10	0.56	0.46
40	8	3.0	$2\frac{1}{16}$	$\frac{1}{4}$	25.5	2.92	0.267	6.5	26.0	33.2	1.00	0.58	0.47
41	7	5.75	$2\frac{9}{16}$	$\frac{5}{8}$	33.5	2.41	0.237	9.5	38.0	49.4	2.29	0.63	0.62
41	7	3.5	$2\frac{1}{4}$	$\frac{5}{16}$	24.0	2.60	0.276	6.75	27.0	35.1	1.47	0.65	0.58
42	7	3.4	$2\frac{1}{8}$	$\frac{11}{32}$	21.0	2.48	0.251	6.0	24.0	31.2	1.08	0.56	0.48
42	7	2.5	2	$\frac{7}{32}$	17.0	2.62	0.281	5.0	20.0	26.0	0.86	0.59	0.50
43	6	5.25	$2\frac{3}{8}$	$\frac{5}{8}$	23.0	2.09	0.243	7.75	31.0	39.9	2.02	0.62	0.65
43	6	3.0	2	$\frac{1}{4}$	16.25	2.33	0.300	5.5	22.0	28.2	1.14	0.62	0.63
44	6	3.0	$1\frac{13}{16}$	$\frac{5}{16}$	14.5	2.21	0.271	4.75	19.0	24.7	0.80	0.51	0.52
44	6	2.25	$1\frac{11}{16}$	$\frac{3}{16}$	12.25	2.33	0.302	4.0	16.0	20.8	0.61	0.52	0.52
45	5	3.9	$2\frac{1}{8}$	$\frac{1}{2}$	12.5	1.79	0.256	5.0	20.0	26.0	1.37	0.59	0.63
45	5	2.6	$1\frac{7}{8}$	$\frac{1}{4}$	9.5	1.93	0.300	3.75	15.0	19.5	0.87	0.58	0.61
46	5	2.675	$1\frac{13}{16}$	$\frac{3}{8}$	8.5	1.77	0.252	3.4	13.6	17.7	0.62	0.48	0.46
46	5	1.70	$1\frac{5}{8}$	$\frac{3}{16}$	6.25	1.92	0.295	2.5	10.0	13.0	0.41	0.49	0.46
47	4	3.15	$2\frac{1}{16}$	$\frac{7}{16}$	7.0	1.47	0.272	3.5	14.0	18.2	1.14	0.60	0.68
47	4	2.4	$1\frac{7}{8}$	$\frac{1}{4}$	5.75	1.55	0.300	2.9	11.6	15.08	0.83	0.59	0.67
48	4	2.25	$1\frac{13}{16}$	$\frac{3}{8}$	4.7	1.44	0.261	2.35	9.4	12.22	0.56	0.50	0.49
48	4	1.5	$1\frac{5}{8}$	$\frac{3}{16}$	3.65	1.56	0.304	1.82	7.3	9.5	0.38	0.50	0.50

CONCENTRATED LOADING.

If there be a concentrated load P on a span l, and dividing the span into two segments, x and $l - x$; then x being the distance from the left support say, $l - x$ is the distance of P from the right support. The reaction at left support is, then, $\frac{P}{l}\left\{l - x\right\}$, and the bending moment is a maximum under the load, and is

$$\frac{P}{l}\left(l - x\right)x = \frac{P}{l}\left(lx - x^2\right) \qquad (1)$$

For a uniformly distributed load of W on the same span l, the maximum bending moment is at the centre, and is given by $\frac{Wl}{8}$. Equating this with $\frac{P}{l}\left(lx - x^2\right)$, we get

$$\frac{Wl}{8} = \frac{P}{l}\left(lx - x^2\right)$$

whence

$$W = 8\,P\left\{\frac{x}{l} - \frac{x^2}{l^2}\right\} \qquad (2)$$

If the concentrated load be at the centre of the span, $x = \frac{l}{2}$, and, substituting this value of x in (2), we get

$$W = 2\,P \qquad (3)$$

Equation (2) gives the equivalent uniformly distributed load W, whose centre bending moment is equal to the maximum moment caused by the load P distant x from left support.

Equation (3) shows that the uniformly distributed load W will cause the same bending moment at centre as the load $P = \frac{W}{2}$, concentrated at the centre of span. In other

words, if a beam of span l sustain, with a given fibre stress, a load, P, concentrated at the centre, it will also sustain, with the same fibre stress, a uniformly distributed load, W, equal to 2 P,—*i.e.*, *double* the load if uniformly distributed.

EXAMPLE. Suppose a load of 8 tons to be concentrated at a point 12 feet from the left support of an 18 feet span. The reaction at the left support $= \frac{P}{l}\left(l - x\right) = \frac{8}{18}\left(18 - 12\right) = \frac{8 \times 6}{18} = 2\frac{2}{3}$ tons. The maximum bending moment is under the load of 8 tons, and is $\frac{P}{l}\left(lx - x^2\right) = \frac{8}{18}\left\{18 \times 12 - \overline{12}^2\right\} = 32$ foot-tons.

From equation (2) we find

$$W = 8\,P\left\{\frac{x}{l} - \frac{x^2}{l^2}\right\} = 8 \times 8 \text{ tons}\left\{\frac{12}{18} - \left(\frac{12}{18}\right)^2\right\}$$

$$= 64 \times \frac{2}{9} = 14.22 \text{ tons.}$$

If the fibre stress is to be 4.15 tons per square inch, and the metal to be of iron, then, as iron beams in Tables of Capacity are figured for 6.0 tons extreme fibre stress, we should look in them for a beam of 18′ span, which has a capacity of $\frac{6}{4.15} \times 14.22 = 20.56$ tons. Looking opposite 18′ spans, we find that a 15″ **I** beam of iron, shape No. 2, 200 pounds per yard, will carry 20.55 tons. This, then, is the beam which will carry a load of 8 tons situated 12′ from the left support of an 18′ span, the fibre stress being 4.15 tons per square inch.

These results could also be obtained in the following way: The maximum bending moment for the concentrated load of 8 tons, 12 feet distant from the left support of an 18′ span, is 32 foot-tons = 384 inch-tons.

Now, $$M_o = f\,.\,\frac{2\,I}{h} = f\,R$$

whence $$R = \frac{M_o}{f}$$

Now, if f be taken 4.15 tons per square inch, then

$$R = \frac{384}{4.15} = 92.53$$

Looking in table of "Properties of I Beams," we find that for R = 92.50, the beam is 15″ I, 200 pounds per yard. This beam, then, will do.

If the concentrated load of 8 tons be at the *centre* of an 18 feet span, the maximum bending moment is under the load, and is $\frac{P\,l}{4} = \frac{8 \times 18}{4} = 36$ foot-tons. The "equivalent" uniformly distributed load is $2\,P = 2 \times 8 = 16$ tons, whose bending moment is $\frac{16 \times 18}{8} = 36$ foot-tons, the same as above. Thus, a beam which will carry 16 tons uniformly distributed, will also carry, at the same fibre stress, a load of 8 tons concentrated at the centre of the span. If the fibre stress is to be $4\frac{1}{4}$ tons, then, in the Tables of Capacity, we must look for an iron beam which has a tabular capacity of $\frac{6}{4\frac{1}{4}} \times 16 = 22.61$ tons at 18′ span. For 18′ span in the tables, we find that a 15″ I beam of iron, 250 pounds per yard, will carry 24.00 tons, which is rather more than we need.

To find the exact weight of a 15″ I beam which will answer our purpose, use the equation $M_o = f\,qh\,S$; whence

$$S = \frac{M_o}{f\,qh}$$

Now $M_o = 36$ foot-tons $= 432$ inch-tons.
f $= 4\frac{1}{4}$ tons, the required extreme fibre stress.
h $= 15''$.
q $= 0.309$ for 15″ I beam, 200 pounds per yard, from table of "Properties of I Beams."

Then

$$S = \text{required area} = \frac{432}{4\frac{1}{4} \times 0.309 \times 15} = 22.00 \text{ square inches};$$

whence we need a 15″ iron I beam, 220 pounds per yard.

If the required fibre stress had been the same as in the tables,—viz., 6.0 tons for iron,—we would have found that, for the given span of 18 feet, the capacity of a 15″ **I** beam of iron, 150 pounds per yard, was 15.66 tons, which is rather less than the 16 tons uniformly distributed load for which we were seeking, and using the same method as before,—viz., the equation $M_o = f\,qh\,S$,—we would have

$$S = \frac{M_o}{f\,qh} = \frac{432}{6.0 \times 0.313 \times 15} = 15.33 \text{ square inches};$$

i.e., we require a 15″ **I** beam of iron, 153½ pounds per yard.

The centre deflexion for a beam under a uniformly distributed load, W, is $\frac{5}{8}$ of that for the same load concentrated at the centre of the span. Inversely, the deflexion for a beam under a concentrated load, P, at centre of span is 1.6 times that for the same load uniformly distributed over the span. As in using the tabular loads to find the beam which will carry a centre concentrated load, we *double* the concentrated load, and seek for a beam to carry such load; then, to find the deflexion for the concentrated load, we must take

$\frac{1.6}{2} = 0.8$ of the tabular deflexion.

Another example. Having given a beam of certain kind, weight, and span, to find what load concentrated at a point x from the left support it can safely carry.

Suppose we have a 12″ iron **I** beam, 125 pounds per yard, on a span of 15 feet. From Tables of Capacity, we find it will carry 12.33 tons, uniformly distributed, the fibre stress being 6.0 tons. Now, what load concentrated at a point distant 4.0′ from the left support will it carry, the fibre stress being the same? From $\frac{W\,l}{8} = \frac{P}{l}\left(lx - x^2\right)$, we get

$$P = \frac{W\,l^2}{8\,(lx - x^2)} = \frac{12.33 \times 15 \times 15}{8\,(15 \times 4 - 16)} = 7.88 \text{ tons};$$

that is, a concentrated load of 7.88 tons, 4 feet from one end, will be carried by the 12″ iron **I** beam, 125 pounds per yard, with the same extreme fibre stress as is produced by 12.33 tons uniformly distributed over the span.

[Written for "Engineering News," in 1884, by J. C. Bland, C.E.]

A Method of Computing the Absolute Maximum Bending Moment on Stringers, due to the Passage across them of a Series of Concentrated Moving Loads.

From an analytical consideration of the effects produced on the stringers of railway bridges by the passage across them of a series of concentrated weights, such as the wheels of a locomotive, the following principles are found to flow:

1. That the maximum bending moment always occurs *under* a load.

2. That the maximum bending moment occurs under one or the other of the two loads, between which the resultant of the total number of loads considered passes.

3. That if the resultant of the total number of loads considered passes through a load, the maximum bending moment occurs under that load.

4. Calling the load under which the maximum bending moment occurs the *critical* load, and x its distance from the left support, then, when the *critical* load is in the position causing the maximum bending moment, its distance from the left support is less than the half span, if the resultant of the total number of loads considered lies to the right of the *critical* load; and greater than the half span, if the resultant lies to the left.

5. Calling Z the distance from the resultant of the total number of loads considered to the load on the right, and Δ the distance apart of the two loads between which the resultant passes, the distance of the load on the left from such resultant is $\Delta - Z$.

Then $$x = \frac{l}{2} + \frac{Z}{2}, \text{ or } x = \frac{l}{2} - \frac{\Delta - Z}{2}$$

according as the *critical* load is on the right or the left of such resultant.

6. Then when the *critical* load is in the position causing the maximum bending moment, the centre of the span divides *equally* the distance between the resultant and the *critical* load, or, in other words, the *critical* load and the resultant of the total number of loads considered are symmetrically placed with reference to the centre of the span.

7. That the expression for the maximum bending moment can always be put in one or the other of the two forms. (*a*) If the *critical* load lies to the right of the resultant of the total number of loads considered,

$$M_o = \frac{\Sigma \,.\, P}{4}\left[l + \frac{Z^2}{l} - 2\left(\frac{\Sigma \,.\, Pd}{\Sigma \,.\, P}\right)\right] \qquad (1)$$

(*b*) If the *critical* load lies to the left of the resultant of the total number of loads considered,

$$M_o = \frac{\Sigma \,.\, P}{4}\left[l + \frac{(\Delta - Z)^2}{l} - 2\left(\frac{\Sigma \,.\, Pd}{\Sigma \,.\, P}\right)\right] \qquad (2)$$

where $\Sigma \,.\, P =$ number of loads on span, expressed in terms of the load on each pair of drivers. For example, if there are loads on the span of less amount than those on the drivers, express them in terms of the driver load. Thus the four pairs of drivers and the first pair of tender wheels, being on the span, express the tender wheel load as a P, whence the total number of loads, $\Sigma \,.\, P = 4\,P + a\,P = (4 + a)\,P$.

Let $l =$ span.

$Z =$ as already defined in 5.

$\Delta - Z =$ as already defined in 5.

$\Sigma \,.\, Pd =$ sum of the moments of loads on span around the *critical* load as origin, *no regard being had as to sign;* that is, no regard being had to the *sense* of the moments.

8. That the expression for Z and for $\Delta - Z$ can always be put in the form

$$\frac{\Sigma^1 .\, Pd}{\Sigma \,.\, P} \qquad (3)$$

where $\Sigma^1 .\, Pd =$ summation of the moments of loads on the

span around the *critical* load as origin, *regard being had to sign;* that is, regard being had to the *sense* of the moments.

9. Whence the maximum bending moment is *always* given by the following general expression:

$$M_o = \frac{\Sigma \,.\, P}{4}\left\{1 + \left[\frac{\Sigma^1 \,.\, Pd}{\Sigma \,.\, P}\right]^2 \cdot \frac{1}{l} - 2\left(\frac{\Sigma \,.\, Pd}{\Sigma \,.\, P}\right)\right\} \quad (4)$$

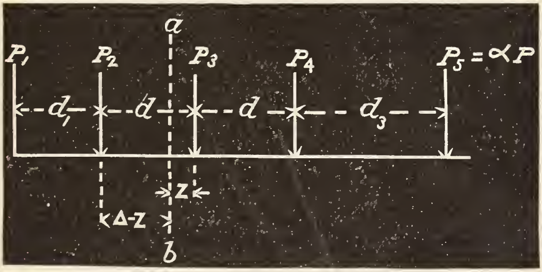
which can be used instead of equations (1) and (2).

10. That is, the cases where the resultant of the total number of loads considered passes between two of the loads, the maximum bending occurring under one or the other of these two loads, then in whichever of the expressions for M_o, considering first one and then the other as the *critical* load, the term

$$\left\{\left(\frac{\Sigma^1 \,.\, Pd}{\Sigma \,.\, P}\right)^2 \cdot \frac{1}{l} - 2\left(\frac{\Sigma \,.\, Pd}{\Sigma \,.\, P}\right)\right\}$$

is the greater, *that one* gives the *absolute* maximum bending moment due to the passage of number of loads considered across the span.

For example: consider the "Erie" consolidation engine, in the case where five loads are on the span,—viz., the four drivers and the first pair of tender wheels.



Where $P_1 = P_2 = P_3 = P_4$, hence call $P. = 11.0$ tons. $P_5 = 7.26$ tons $= a\,P$; whence

$$a = \tfrac{726}{1100} = 0.66$$

$$d = 4.5 \text{ feet},\ d_1 = 5.75 \text{ feet},\ d_3 = 7.083 \text{ feet}.$$

From these values it is found that the line of action of the resultant of these loads passes between P_2 and P_3; whence by 2, the maximum bending moment will occur under the loads P_2 or P_3. Let us first consider P_3 as the *critical* load, and apply our equation (4). Then

$$\Sigma . P = 4\,P + a\,P = (4 + a)\,P . = 4.66\,P .$$

Taking moments of loads around P_3 as an origin, we have on the right of P_3,

$$P_4\,d + P_5\,(d + d_3) = P\left\{d + a\,(d + d_3)\right\}$$

on the left of P_3,

$$P_2\,d + P_1\,(d + d_1) = P\left\{2\,d + d_1\right\}$$

now counting moments whose tendency is opposite to the hands of a watch as positive, and those whose tendency is same as the hands of a watch as negative, then the moments on the left of P_3 are positive, and those on the right of P_3 are negative; whence

$$\Sigma^1 . Pd = P\left\{2\,d + d_1 - \left[d + a\,(d + d_3)\right]\right\}$$

$$= P\left\{d + d_1 - a\,(d + d_3)\right\}$$

also, as $\Sigma . Pd$ is sum of moments without regard to the sign, we have

$$\Sigma . Pd = P\left\{2\,d + d_1 + d + a\,(d + d_3)\right\}$$

$$= P\left\{3\,d + d_1 + a\,(d + d_3)\right\}$$

Then

$$\frac{\Sigma^1 . Pd}{\Sigma . P} = \frac{d + d_1 - a\,(d + d_3)}{4 + a}$$

and

$$2\left(\frac{\Sigma . Pd}{\Sigma . P}\right) = 2\left\{\frac{3\,d + d_1 + a\,(d + d_3)}{4 + a}\right\}$$

Substituting these values in equation (4), we get

$$M_o = \frac{4+a}{4} P \left\{ l + \left[\frac{d + d_1 - a\,(d + d_3)}{4+a}\right]^2 \cdot \frac{1}{l} \right.$$

$$\left. - 2 \left[\frac{3\,d + d_1 + a\,(d + d_3)}{4+a}\right] \right\}$$

Inserting in the above the values given for the distances between loads, etc., we get

$$M_o = \frac{4.66}{4} \times 11.0 \left\{ l + \frac{\overline{.559}^2}{l} - 11.543 \right\}$$

$$= 12.815 \left\{ l + \frac{.3125}{l} - 11.543 \right\}$$

$$= 12.815\, l + \frac{4.005}{l} - 147.923$$

Now, suppose our span is 30 feet; then

$$M_o = 12.815 \times 30 + \frac{4.005}{30} - 147.923 = 236.66 \text{ foot-tons.}$$

Let us now take another case at random, say the three pairs of drivers, P_1, P_2, P_3. The distance d_1 being generally greater than d, and the driver loads alike, it is evident the line of action of the resultant will pass between the loads P_1 and P_2. It is the case in the "Erie" engine we are considering for illustrations. Let us take P_2 as the *critical* load; then

$$\Sigma\,.\,P = 3\,P.$$

$$\Sigma^1\,.\,Pd = P\,(d_1 - d).$$

$$\Sigma\,.\,Pd = P\,(d_1 + d).$$

Whence $$\frac{\Sigma^1\,.\,Pd}{\Sigma\,.\,P} = \frac{d_1 - d}{3}$$

and $$2\left(\frac{\Sigma\,.\,Pd}{\Sigma\,.\,P}\right) = \frac{2}{3}\left(d_1 + d\right)$$

whence

$$M_o = \frac{3\,P}{4}\left\{l + \left(\frac{d_1 - d}{3}\right)^2 \frac{1}{l} - \frac{2}{3}\left(d_1 + d\right)\right\}$$

Inserting the values given for the load, P, and the distances, d and d_1, we get

$$M_o = \frac{3 \times 11}{4}\left\{l + \frac{0.1736}{l} - 6.833\right\}$$

$$= 8.25\,l + \frac{1.432}{l} - 56.375$$

Now, suppose the span is 15 feet; then

$$M_o = 8.25 \times 15 + \frac{1.432}{15} - 56.375 = 67.47 \text{ foot-tons.}$$

If we had chosen the three driver loads, P_2, P_3, P_4, we see that the resultant passes through the load P_3, since the other two loads are equally distant, d, from it; whence the *critical* load is P_3. Here, then,

$$\begin{aligned} \Sigma\,.\,P &= 3\,P. \\ \Sigma^1\,.\,Pd &= P\,(d - d) = 0. \\ \Sigma\,.\,Pd &= P\,(d + d) = 2\,Pd. \end{aligned}$$

Whence $\dfrac{\Sigma^1\,.\,Pd}{\Sigma\,.\,P} = 0$ and $2\left(\dfrac{\Sigma\,.\,Pd}{\Sigma\,.\,P}\right) = \dfrac{4}{3}\,d$

whence $M_o = \dfrac{3\,P}{4}\left\{l - \dfrac{4}{3}\,d\right\}$

Inserting the values for P and d, we get

$$M_o = \frac{3 \times 11}{4}\left\{l - 6\right\} = 8.25\,l - 49.5$$

Now, suppose the span to be 15 feet; then

$$M_o = 8.25 \times 15 - 49.5 = 74.25$$

In passing, we might notice that this choice of loads gives a greater result than the loads P_1, P_2, P_3.

Let us now take the four drivers, P_1, P_2, P_3, P_4. It is readily seen that in usual cases the resultant of the four loads pass between the loads P_2 and P_3.

Let us take the load P_3 as the *critical* load; then we have

$$\Sigma . P = 4 P.$$
$$\Sigma^1 . Pd = P (d + d + d_1) - Pd = P (d + d_1).$$
$$\Sigma . Pd = P (d + d + d_1) + Pd = P (3 d + d_1).$$

Whence $$\frac{\Sigma^1 . Pd}{\Sigma . P} = \frac{d + d}{4}$$

and $$2 . \left(\frac{\Sigma . Pd}{\Sigma . P}\right) = 2 \left(\frac{3 d + d_1}{4}\right) = \frac{3 d + d_1}{2}$$

whence

$$M_o = \frac{4 P}{4} \left\{ l + \left(\frac{d + d_1}{4}\right)^2 \frac{1}{l} - \frac{3 d + d_1}{2} \right\}$$

$$= P \left\{ l + \left(\frac{d + d_1}{4}\right)^2 . \frac{1}{l} - \frac{3 d + d_1}{2} \right\}$$

Inserting the values of P, d and d_1 for the "Erie" engine, we get

$$M_o = 11 \left\{ l + \frac{6.566}{l} - 9.625 \right\} = 11 \, l + \frac{72.23}{l} - 105.875$$

Suppose the span to be 21 feet; then

$$M_o = 11 \times 21 + \frac{72.23}{21} - 105.875 = 128.56 \text{ foot-tons.}$$

Let us now take P_2 as the *critical* load. We then get

$$\Sigma^1 . Pd = Pd_1 - P (d + 2 d) = - P (3 d - d_1)$$

and

$$\Sigma . Pd = Pd_1 + P (d + 2 d) = P (3 d + d_1)$$

whence

$$\frac{\Sigma^1 . Pd}{\Sigma . P} = - \frac{3 d - d_1}{4} \text{ and } 2 \left(\frac{\Sigma . Pd}{\Sigma . P}\right) = \frac{3 d + d_1}{2}$$

Now

$$\left(\frac{\Sigma^1 . \mathrm{Pd}}{\Sigma . \mathrm{P}}\right)^2 = \left(-.\frac{3\,\mathrm{d}-\mathrm{d}_1}{4}\right)^2 = \left(\frac{3\,\mathrm{d}-\mathrm{d}_1}{4}\right)^2$$

whence

$$\mathrm{M}_0 = \mathrm{P}\left\{1 + \left(\frac{3\,\mathrm{d}-\mathrm{d}_1}{4}\right)^2 . \frac{1}{\mathrm{l}} - \frac{3\,\mathrm{d}+\mathrm{d}_1}{2}\right\}$$

Inserting the values of P, d and d_1 for the "Erie" engine, we get

$$\mathrm{M}_0 = 11.0\left\{\mathrm{l} + \frac{3.754}{\mathrm{l}} - 9.625\right\}$$

$$= 11\ \mathrm{l} + \frac{41.294}{\mathrm{l}} - 105.875$$

Computing for a 21 feet span, we get

$$\mathrm{M}_0 = 11 \times 21 + \frac{41.294}{21} - 105.875 = 127.09 \text{ foot-tons.}$$

It is noticed that this result is less than that given by choosing P_3 as the *critical* load.

Sufficient illustrations have been given to show how easy of application is the general expression (4).

When any number of loads are considered, the two loads between which the resultant passes can generally be determined by inspection,—if not easily seen, the determination is readily found. Then apply the expression (4), first considering the load on one side, then the load on the other side of the resultant as the *critical* load. Whichever gives the greater value of M_0, is the expression to use in computing the bending moments for that number of loads within the limits of span, both superior and inferior. Considering any particular engine, a table can be calculated showing the bending moments and limits of span for one, two, three, four, five, etc., loads in succession.

ON THE USE OF

THE TABLES OF CAPACITY.

In the table showing the reduction of extreme fibre stresses due to ratio of flange length to flange width, we notice that for fifty ratios the extreme fibre stress for *steel* shapes is reduced to 6.07 tons per square inch, which is very nearly that for which the capacity of the iron shapes has been calculated.

If, then, when we find that the tabular safe load of an iron shape would fulfil the requirements, but, by reason of the beam being *unstayed*, we have to reduce its load to 77 per cent. of its tabular capacity, we can substitute the *steel* shape of the same sectional area, and *all* our requirements are satisfied.

For example: Take a 15″ iron **I** beam, 150 pounds per yard, at 21′ span. Its tabular capacity is 13.43 tons; but its ratio of length to flange width $= \frac{21' \times 12''}{5''} = 50.4$; whence its fibre stress should be 4.64 tons, instead of 6.0 tons, and hence it will carry but 0.773 of its tabular capacity, —viz., only $0.773 \times 13.43 = 10.38$ tons. Now, looking at the same shape in *steel*, we see its tabular capacity is 17.46 tons, and the ratio of its unstayed length to flange width being as before, the reduced safe load will be $0.773 \times 17.46 = 13.50$ tons.

Thus it is seen that the *steel* **I** beam, which has 15.0 square inches sectional area, will carry, when *unstayed* its full length of 21.0 feet, the same load which the iron **I** beam of same sectional area would carry if *stayed*, so that

its unsupported length of flange was no greater than 30 times its flange width. The limit to the 15″ iron **I** beam, in order to use the tabular loads, would be $30 \times 5'' = 150'' = 12\frac{1}{2}'$, —*i.e.*, in order to use a fibre stress of 6.0 tons per square inch; and the steel **I** beam, *unstayed* for its full length, could be used at the same extreme fibre stress of 6.0 tons.

These facts are of use in designing the floor joist of a building, for frequently, by simply substituting steel shapes of same sectional areas as the iron ones, and which weigh only a little more per foot, we can do away with the necessity of some method of *staying* the flanges, or of having to use much heavier beams of iron.

It is also to be remembered that steel beams and channels cost no more *per pound* than iron ones; whence any saving in weight by the use of steel shapes is a like saving in cost.

Suppose the area of a floor surface to be $20' \times 28'$, and we desire to find the beam requisite to carry a total loading of 200 pounds per square foot. We would, of course, place the beams with their length in smaller dimension of the floor area; then the span centre to centre of the beams will be about 21 feet. Suppose, also, that by reason of using brick arches between the beams to carry the external floor load the distance apart of the beams is limited to 5′.0″.

Examining our Tables of Capacity of Iron **I** Beams, we find that a 12″ **I** beam, 125 pounds per yard, shape No. 6, might answer; as for 21 feet span, and 200 pounds per square foot, the distance apart should not be greater than 4.19 feet. But the flange width is $4\frac{7}{8}''$, and the ratio of 21 feet to flange width is 52; whence this exceeding the ratio 30, the extreme fibre stress must be reduced from the tabular amount—viz., 6.0 tons—to about 4.5 tons; in other words, the safe load from 8.81 tons—the tabular safe load—to $0.75 \times 8.81 = 6.61$ tons, and likewise the distance apart will be now $0.75 \times 4.19 = 3.14$ feet. Now, this distance will be too close for the beams, so we should have to select another shape.

Looking at span 21 feet under 12″ iron **I** beam, 170 pounds, shape No. 4, we find that for 200 pounds per square foot, the spacing may be 5.53 feet. The ratio of length to

flange width is $\frac{21 \times 12}{5\frac{3}{8}''} = 47$; whence the distance 5.53 should be reduced to about $0.8 \times 5.53 = 4.42$ feet. We might make six spaces of 4′.8″ in the 28 feet length of floor, and hence would require five 12″ **I** beams of iron, 170 pounds per yard, 21′.6″ long each, weighing in all 6090 pounds. Now, looking at a steel 12″ **I** beam of $126\frac{1}{4}$ pounds per yard (12.50 square inches area), we find that for 21 feet span, under the tabular loads, it may be spaced 5.46 feet. But the ratio of length of beam to flange width being $\frac{21 \times 12}{4\frac{11}{16}''} = 54$, the distance can only be $0.74 \times 5.46 = 4.04$ feet.

Making seven spaces in the 28.0 feet, of 4′.02″ each, we require 6 steel **I** beams, $126\frac{1}{4}$ pounds per yard, 21′.6″ long each, weighing in all 5430 pounds. Thus, even with *one* more beam, by using the steel, we save a weight of 660 pounds, or about 11 per cent.; and this is also a saving in cost of 11 per cent., because steel beams and channels cost no more per pound than do iron ones.

Suppose we have a floor area $18' \times 32'$, and a total floor load of 200 pounds per square foot, and that we wish to make 4.0 feet spaces between centres of beams. Placing the beams in short way of floor area, they will be 19 feet span centre to centre of bearings; and in 32 feet of length we will have eight spaces of 4 feet each, or require 7 beams, say $19\frac{1}{2}$ feet long each.

Assuming the flange width about $4\frac{1}{2}'' = \frac{3}{8}$ of a foot, if beams are *unstayed* laterally, the ratio of unstayed flange to flange width will be $18 \div \frac{3}{8} = 48$; whence, by looking at Table of Reduction of Fibre Stresses and Tabular Loads, we see that tabular capacity will have to be multiplied by about 0.8, and tabular spacing also by 0.8; whence, in order to use the Tables of Capacity, if we divide the required spacing by 0.8, it will give us a spacing which, if we find the corresponding beams in the tables, they will fulfil our conditions. Thus, $\frac{4.00}{0.8} = 5.00$ feet. Now, looking in Tables of

Iron **I** Beams, at 19 feet spans, we find, under column of 200 pounds per square foot, that a $10\frac{1}{2}$ **I** beam of iron, 135 pounds per yard, will carry 9.58 tons, and be spaced 5.02 feet apart. Now, flange width of $10\frac{1}{2}$ **I**, 135 pounds, is 5″; whence ratio of unstayed length to flange width is $\frac{19 \times 12}{5} = 46$; then tabular safe load and tabular spacing will have to be multiplied by about .81. Thus, tabular load $\times\ 0.81 = 9.58 \times 0.81 = 7.75$ tons; and tabular spacing $\times\ 0.81 = 5.02' \times 0.81 = 4.06$ feet; that is, we can use $10\frac{1}{2}''$ **I** beams of iron, and spacing them 4.0 feet apart will compensate for the reduction of capacity due to beams being *unstayed.* We found the reduced safe load to be for this beam 7.75 tons, and this will be seen to be right, for the load to be carried is $19' \times 4'$ apart $\times$ 200 pounds per square foot $=$ 15,200 pounds $=$ 7.60 tons; whence weight is $7 - 10\frac{1}{2}''$ **I** beams (iron), 135 pounds per yard, $19\frac{1}{2}'$ long $=$ 6142 pounds.

To see what steel beam will satisfy the conditions. The spacing which we wish to use is 4.0 feet, and in Tables of Steel **I** Beams we find for a 19 feet span and 200 pounds per square foot of load, that the spacing is 4.14 feet, and load carried 7.87 tons, but, bearing in mind the reduction of strength by reason of beams not being stayed, we should look in the steel tables for a beam which will have a spacing under the 200 pounds column of $\frac{4.0}{0.8} = 5.0'$, and a load of $\frac{7.60}{0.8} = 9.50$ tons. The nearest to this is a $10\frac{1}{2}''$ **I** beam of steel, 106 pounds per yard, shape No. 9, 9.18 tons safe load, and 4.83 feet spacing.

It is evident that a little increase of section in this beam would add enough to strength so as to make it answer our purpose.

To find what weight of this shape we would need, we have from Table of Properties of **I** Beams, $q = 0.301$, say 0.30, and using equation (20), page 157, we have

$$S = \frac{M_o}{f\ qh}$$

Now $M_o = \frac{Wl}{8} = \frac{7.60 \times 19 \times 12}{8} = 216.6$ inch-tons.

$f = 0.80 \times 7.8 = 6.04$ tons per square inch.

$q = 0.30.$

$h = 10\frac{1}{2}''.$

Then area required $= S = \frac{216.6}{6.04 \times 0.3 \times 10\frac{1}{2}} = \frac{216.6}{19.03} =$ 11.38 square inches.

Or a $10\frac{1}{2}''$ **I** beam (*steel*) of shape No. 9, and weighing 115 pounds per yard (11.38 square inches area).

Now from (18), page 156,

$$R = qh\ S = 0.3 \times 10\tfrac{1}{2} \times 11.38 = 35.85$$

whence safe load for steel beams (see equation (11), page 152) is

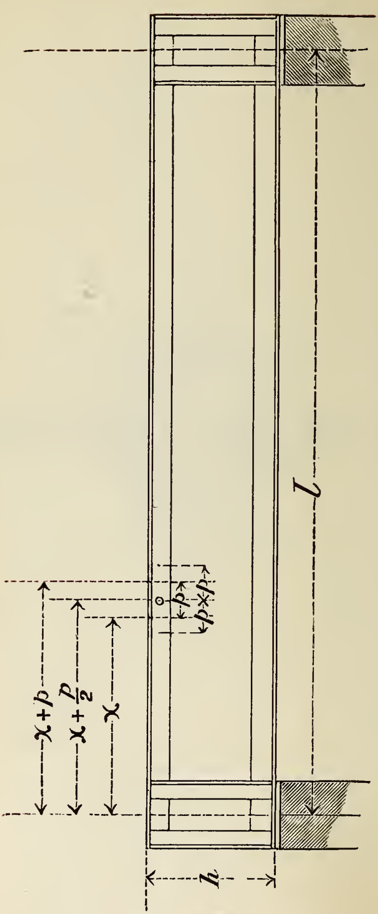
$$W = \frac{5.2\,R}{l'} = \frac{5.2 \times 35.85}{19} = 9.81 \text{ tons,}$$

and reducing this by multiplying by 0.8, we get $9.81 \times 0.8 = 7.85$ tons as the safe load, when beam is *unstayed* in its length of 19 feet.

Then for the weight of the steel beams, 7 beams, $10\frac{1}{2}''$ **I** steel, 115 pounds per yard, $19\frac{1}{2}$ feet long $=$ 5232 pounds.

Now $10\frac{1}{2}''$ **I** iron beams, 135 pounds per yard, weighed for the 7 of $19\frac{1}{2}$ feet each, 6142 pounds; whence a saving of 910 pounds in the floor joist, or almost 15 per cent., likewise a saving of 15 per cent. in cost.

ON PLATE GIRDERS.



x+p
x+p/2
x
p
p×p
l
h

PLATE GIRDERS.

Let l = span, centre to centre of end bearings.
h = height of girder, centre to centre of gravity of flanges.
} Both in same linear units.

w = load per linear unit of span.

R_a = reaction at left abutment, a.

R_{a^1} = reaction at right abutment, a^1.

F_x = shear at section distant x from left abutment.

F_o = shear at end of girder = maximum shear.

M_x = bending moment at section distant x from left abutment.

$\frac{M_x}{h}$ = flange stress at section distant x from left abutment.

f_c = allowable stress per square inch in compression.

$p_c = \dfrac{f_c}{1 + \dfrac{1}{5000}\left(\dfrac{l}{w}\right)^2}$ = reduced compression unit, due to length of unstayed portion of upper flange as regards its width.

w = flange width

f_t = allowable stress per square inch in tension.

f_{ps} = allowable shearing stress per square inch on the web plates.

f_{rs} = allowable shearing stress per square inch on rivets.

f_{rb} = allowable bearing stress per square inch on rivets.

$\square_c'' = \dfrac{M_x}{p_c h}$ = gross sectional area required in upper flange at centre of span.

$\square_t'' = \dfrac{M_x}{f_t h}$ = nett sectional area required in lower flange at centre of span.

The *bending moment*, M_x, at a section distant x from the left abutment, is the *algebraic sum* of the moments around x, of all the external forces acting between the left abutment and the section x.

The shear, F_x, at a section distant x from the left abutment, is the *algebraic sum* of all the external forces acting between the left abutment and the section x.

Plate girder under a uniformly distributed load, w, per linear unit.

Then $$M_x = \frac{wl}{2}x - \frac{wx^2}{2} = \frac{wx}{2}\left(l - x\right) \qquad (1)$$

$$F_x = \frac{wl}{2} - wx = w\left(\frac{l}{2} - x\right) \qquad (2)$$

also

$$M_{(x+p)} = \frac{wl}{2}\left(x + p\right) - \frac{w\,(x+p)^2}{2}$$

$$= \frac{w\,(x+p)}{2}\left(l - x - p\right) \qquad (3)$$

$$F_{(x+p)} = \frac{wl}{2} - w\left(x + p\right) = w\left\{\frac{l}{2} - \left(x + p\right)\right\} \qquad (4)$$

The shear at any point x is the differential coefficient of the bending moment M_x at the point x, and equations (2) and (4) could be derived directly from (1) and (3). Thus,

$$F_x = \frac{d}{dx}\left\{M_x\right\} = \frac{d}{dx}\left\{\frac{wx}{2}\left(l - x\right)\right\} = \frac{wl}{2} - wx \quad (5)$$

and

$$F_{(x+p)} = \frac{d}{dx}\left\{M_{(x+p)}\right\} = \frac{d}{dx}\left\{\frac{w\,(x+p)}{2}\left(l - x - p\right)\right\}$$

$$= \frac{wl}{2} - w\left(x + p\right) \qquad (6)$$

Now, flange stress at point x is

$$\frac{M_x}{h} = \frac{wx}{2h}\left(l - x\right) \qquad (7)$$

and flange stress at point x + p is

$$\frac{M_{(x+p)}}{h} = \frac{w(x+p)}{2h}\left(l - x - p\right) \qquad (8)$$

The difference of these flange stresses is the stress on the rivets in the distance p,—*i.e.*,

$$\frac{M_{(x+p)}}{h} - \frac{M_x}{h} = \frac{w}{2h}\left\{pl - 2px - p^2\right\}$$

$$= \frac{wp}{2h}\left\{l - 2x - p\right\} = \left\{\frac{wl}{2} - w\left(x + \frac{p}{2}\right)\right\}\frac{p}{h} \qquad (9)$$

But shear at the section distant $\left(x + \frac{p}{2}\right)$ from left abutment is

$$F_{\left(x+\frac{p}{2}\right)} = \frac{wl}{2} - w\left(x + \frac{p}{2}\right)$$

whence equation (9) could be written

$$\frac{M_{(x+p)}}{h} - \frac{M_x}{h} = F_{\left(x+\frac{p}{2}\right)} \cdot \frac{p}{h} \qquad (10)$$

that is, the stress on the rivets in the distance p, is the shear at the *middle* of the distance p, multiplied by the ratio of the distance p to the height h. Or, if the distance p be the pitch of the rivets, the stress on the rivet is the shear at that rivet multiplied by the pitch and divided by height of girder.

Thus, generally, calling a the stress on a rivet distant x from the abutment,

$$a = \frac{F_x \cdot p}{h} \qquad (11)$$

i.e., stress on rivet at section x is the shear at x multiplied by the pitch and divided by height of girder.

From (11) we get

$$p = \frac{h\,a}{F_x} = \frac{a}{\frac{1}{\frac{F_x}{h}}} \qquad (12)$$

that is, the pitch of the rivet at any section x is the allowable stress on the rivet multiplied by the height of girder and divided by the shear at the rivet.

If we take the stress on the rivets in a distance, h, equal to the height of the girder, and say n the number of rivets in such distance; then

$$n\,a = F_{\left(x + \frac{h}{2}\right)} \qquad (13)$$

that is, the number of rivets in the distance h, multiplied by the mean stress on each rivet, is the shear at a point distant $\left(x + \frac{h}{2}\right)$ from the abutment.

If, in (13), we make $x = 0$, then the stress on the rivets in the distance from the abutment to the section h—that is, in a distance from end of girder equal to height—is $F_{\left(\frac{h}{2}\right)}$. In other words,

$$n\,a = \frac{wl}{2} - \frac{wh}{2} = \frac{w}{2}\left(l - h\right) \qquad (14)$$

Now, $\frac{wl}{2} - \frac{wh}{2}$ is the flange stress at the point h; whence

$$n\,a = \frac{M_h}{h} \qquad (15)$$

i.e., the *entire* flange stress at a point whose distance from the abutment is equal to the depth of girder, must be conveyed to the flange angles by means of the rivets which connect the flange angles to the web.

But we must bear in mind that from 0 to h the flange stress increases from 0 to $\frac{M_h}{h}$, and if we proportioned the number of rivets by (14) and (15), a would be the *mean* stress on the rivets in the distance h; we should, however,

determine the number from the *maximum* stress in the distance h,—that is to say, in (14) make h = o, and then $n\,a = \frac{wl}{2}$. In other words, the number of rivets required in a distance from end supports equal to depth of the girder is

$$n = \frac{F_o}{a} \qquad (16)$$

where F_o is the end shear, which is equal to the reaction, and a the allowable stress on the rivet.

If we divide both members of the above equation by h, the height in feet, then

$$\text{number of rivets per foot} = \frac{\frac{F_o}{h}}{a} = \text{shear per foot,}$$

divided by allowable stress on the rivet.

Now, considering the connexion of the two flange angles to the web sheet, the rivet may be sheared out between the angles, or it may crush the bearing on the web sheet. The stress on the rivet must then *not* exceed its shearing value nor its bearing value. The rivet being in double shear,—*i.e.*, there being two shearing areas, one on each side of the web,—its shearing value is 2 $f_{rs}a$, where f_{rs} is the allowable shearing stress per square inch on rivets, and a the area of the rivet. The rivet having a bearing on the web sheet of dt, where d is the diameter of the rivet, and t the thickness of the web, its bearing value is $f_{rb}dt$, f_{rb} being the allowable bearing stress per square inch; whence a must not exceed 2 $f_{rs}a$, nor $f_{rb}dt$,—*i.e.*,

$$a = \frac{F_x \cdot p}{h} \leqq 2\ f_{rs}a, \text{ and } \leqq f_{rb} \cdot dt$$

whence for shearing,

$$\text{area of rivet, } a = \frac{F_x \cdot p}{2\ f_{rs}h} \qquad (17)$$

and for bearing,

$$\text{thickness of plate, } t = \frac{F_x \cdot p}{f_{rb} \cdot hd} \qquad (18)$$

TABLE OF SHEARING VALUE OF RIVETS

For allowable units of from 3.0 to 4.0 tons per square inch.

Diam. of rivet, d.	Area of rivet, □″.	Value of rivets in single shear at the following allowable shearing units = f_{rs} □″.					
		3.0 tons per square in.	3.25 tons per square in.	3.50 tons per square in.	3.75 tons per square in.	4.0 tons per square in.	4.5 tons per square in.
½″	0.1963	0.59	0.64	0.69	0.74	0.79	0.88
9/16″	0.2485	0.74	0.81	0.87	0.93	0.99	1.12
⅝″	0.3068	0.92	1.00	1.07	1.15	1.23	1.38
11/16″	0.3712	1.11	1.21	1.30	1.39	1.48	1.67
¾″	0.4417	1.33	1.44	1.54	1.66	1.77	1.99
13/16″	0.5185	1.56	1.69	1.81	1.94	2.07	2.33
⅞″	0.6013	1.80	1.95	2.10	2.25	2.40	2.70
15/16″	0.6903	2.07	2.24	2.42	2.59	2.76	3.16
1″	0.7854	2.36	2.55	2.75	2.94	3.14	3.53
1 1/16″	0.8866	2.66	2.88	3.10	3.32	3.55	3.99
1⅛″	0.9940	2.98	3.23.	3.48	3.73	3.98	4.47

TABLE OF BEARING VALUE OF RIVETS

For allowable units of 6.0, 7.5, and 9.0 tons per square inch.

Thickness of plate, t.	Bearing value for different thicknesses of plates = $f_{rb} \times d \times t$.											
	Bearing unit f_{rb}=6.0 tons.				Bearing unit f_{rb}=7.5 tons.				Bearing unit f_{rb}=9.0 tons.			
	Diameter of rivet, d.				Diameter of rivet, d.				Diameter of rivet, d.			
	½″	⅝″	¾″	⅞″	½″	⅝″	¾″	⅞″	½″	⅝″	¾″	⅞″
¼″	0.75	0.94	1.13	1.31	0.94	1.17	1.41	1.65	1.13	1.41	1.69	1.96
5/16″	0.94	1.17	1.41	1.64	1.17	1.46	1.76	2.05	1.41	1.76	2.11	2.46
⅜″	1.13	1.41	1.69	1.97	1.41	1.76	2.11	2.46	1.69	2.11	2.53	2.95
7/16″	1.31	1.64	1.97	2.30	1.64	2.05	2.46	2.87	1.97	2.46	2.95	3.44
½″	1.50	1.88	2.25	2.63	1.88	2.34	2.81	3.28	2.25	2.81	3.38	3.94
9/16″	1.69	2.11	2.53	2.95	2.11	2.64	3.16	3.69	2.53	3.16	3.80	4.43
⅝″	1.88	2.34	2.81	3.28	2.34	2.93	3.52	4.10	2.81	3.52	4.22	4.92
11/16″	2.06	2.58	3.09	3.61	2.58	3.22	3.87	4.51	3.09	3.87	4.64	5.41
¾″	2.25	2.81	3.38	3.94	2.81	3.52	4.22	4.92	3.38	4.22	5.06	5.90

The thickness of web of a girder is generally limited to $\frac{3}{8}$ of an inch for practical reasons; and, besides filling the condition $\frac{\text{maximum shear}}{f_{ps}}$, it must also resist the tendency to buckling; that is, the unit stress on the web should be determined by

$$p_{ps} = \frac{5.00 \text{ tons}}{1 + \frac{1}{3000}\left(\frac{h}{t}\right)^2} \qquad (19)$$

P 2.15
2 3

The girder should be divided into panels by the use of stiffening angle iron on the web sheet, and the length of such panels should generally be about the depth of the girder, unless the girder be quite shallow, in which case the panels may be about one and one-half times the depth.

In equation (19) it is allowable to consider h as the vertical distance *in the clear* between the angle iron flanges.

The permissible unit stresses on plate girders are determined from the following relations, where ϕ denotes the ratio of the minimum stress to the maximum stress.

Compressive unit stress, $f_c = 1\frac{2}{3}$ tons $(2 + \phi)$. (*a*)
Tensile unit stress, $f_t = 2$ tons $(2 + \phi)$. (*b*)
Shearing stress on web plate, $f_{ps} = 1\frac{2}{3}$ tons $(2 + \phi)$. (*c*)
Shearing stress on rivets, $f_{rs} = 1\frac{1}{2}$ tons $(2 + \phi)$. (*d*)
Bearing stress on rivets, $f_{rb} = 3$ tons $(2 + \phi)$. (*e*)

In plate girders under uniformly distributed loads the stresses are in same ratios as the loads, and ϕ may then denote the ratio of the *dead* load to the *total* load.

In plate girders used in buildings and warehouses the loads are all dead, and then ϕ becomes unity, and the above permissible unit stresses become

$f_c = 5.00$ tons per square inch on gross area.
$f_t = 6.00$ tons per square inch on nett area.
$f_{ps} = 5.00$ tons per square inch on nett area.
$f_{rs} = 4.50$ tons per square inch on rivet area.
$f_{rb} = 9.00$ tons per square inch on bearing area of rivet.

Taking f_t as a unit of comparison, the expressions (*a*), (*b*), (*c*), (*d*), (*e*) are in the following ratios:

$$f_c = \tfrac{5}{6} f_t.$$
$$f_{ps} = f_c = \tfrac{5}{6} f_t.$$
$$f_{rs} = \tfrac{3}{4} f_t.$$
$$f_{rb} = 2\, f_{rs} = 1\tfrac{1}{2} f_t.$$

And, taking f_c as a unit of comparison, we get

$$f_t = \tfrac{6}{5} f_c.$$
$$f_{ps} = f_c.$$
$$f_{rs} = 0.9\, f_c.$$
$$f_{rb} = 2\, f_{rs} = 1.8\, f_c.$$

EXAMPLE I.

SINGLE-WEBBED PLATE GIRDER.

Suppose we have a girder 32′ 0″ long, centre to centre of end bearings, and it is required to carry 128 tons uniformly distributed over its length. Dividing the span into eight panels of 4′ 0″ each; at each panel point we will use a pair of angle iron stiffeners, one on each side of the web. We will make the girder 40″ deep out to out of flange angles, which will be the effective depth in this case, as when the flange plates are considered, the 40″ will be about the distance centres of gravity of the flange areas.

Our unit stresses are $f_c = 5.00$ tons; $f_t = 6.00$ tons; $f_{ps} = 5.0$ tons; $f_{rs} = 4.50$ tons; $f_{rb} = 9.0$ tons; and using 14″ flange plates, the ratio of length to width of flange (supposing the flange *unstayed* in its length) will be $32 \div 1\tfrac{1}{6} = 27.43$, whence compressive unit stress f_c is reduced to

$$p_c = \frac{5.00}{1 + \frac{1}{5000}\left(27.43\right)^2} = \frac{5.00}{1.150} = 4.35 \text{ tons.}$$

This will be the maximum permissible unit stress on the upper flange.

We then have given

l = span centre to centre of end bearings = 32 feet.
h = effective height = 40″ = $3\frac{1}{3}$ feet.
w = load per linear foot = $\frac{128}{32}$ = 4.0 tons.

Then bending moment at any point x from left abutment is given by

$$M_x = \frac{wx}{2}\left(l - x\right) = \frac{4.0}{2}x\left(32 - x\right) = 2.0\left(32x - x^2\right)$$

For bending moment at centre of span we have

$$x = \frac{l}{2} \text{ in the equation } M_x = \frac{wx}{2}\left(l - x\right)$$

i.e.,

$$M_c = \frac{wl^2}{8} = \frac{4.0 \times 32 \times 32}{8} = 512 \text{ ft.-tons} = 6144 \text{ in.-tons.}$$

Whence flange stress at centre of span is

$$\frac{M_c}{h} = \frac{6144}{40} = 153.60 \text{ tons.}$$

The flange section required at centre of span to resist compression is

$$\frac{M_c}{p_c h} = \frac{6144}{4.35 \times 40''} = 35.31 \text{ square inches gross.}$$

The flange section required at centre of span to resist tension is

$$\frac{M_c}{f_t h} = \frac{6144}{6.0 \times 40''} = 25.60 \text{ square inches nett.}$$

For compression flange—*i.e.*, for upper flange—use

	Sq. in.
2 angle irons, 6″ × 4″ × $\frac{1}{2}$″, 48 pounds per yard =	9.60
5 flange plates, 14″ × $\frac{3}{8}$″ =	26.25
Total gross section used in upper flange =	35.85

For tension flange—*i.e.*, the lower flange—use

		Sq. in., nett.
2 angles, $6'' \times 4'' \times \frac{1}{2}''$, 48 pounds per yard =	9.60	
Deduct 4 holes, $1''$ diameter $\times \frac{1}{2}''$ thick =	2.00 =	7.60
4 plates, $14 \times \frac{3}{8}$ =	21.00	
Deduct 4 (2 holes, $1'' \times \frac{3}{8}''$) =	3.00 =	18.00
Total nett section used in lower flange =		25.61

In deducting for rivet holes in the tension flange to get the *nett area*, the rivet holes are taken $\frac{1}{8}''$ larger than diameter of the rivet. In above we have assumed $\frac{7}{8}''$ rivets; whence holes are taken $1''$ diameter.

Having now determined the sections to be used at the centre of span, the next step is to find where the several flange plates begin and end,—*i.e.*, the lengths of the various flange plates. The pair of flange angles and the *first* flange plate (the first flange plate is the one *next* the flange angles) extend from end to end of girder, and the other flange plates should extend about two rivet pitches beyond the points where they should stop theoretically. In order to determine these points, we take the general equation for the section required at any point distant x from left abutment,—viz.,

$$\frac{M_x}{p_c h} = \frac{wx}{2\, p_c h}\left(l - x\right) = \frac{4.0\,x\,(32 - x)}{2 \times 4.35 \times 3\frac{1}{3}}$$

$$= \frac{32\,x - x^2}{7.25} = \frac{4}{29}\left\{32\,x - x^2\right\}$$

i.e., square inches required at any point x of the girder = $\frac{4}{29}\left\{32\,x - x^2\right\}$ where x is taken in feet. Now, to find the point where the *second* flange should begin, equate the areas of the two flange angles and *first* flange plate,—viz., 9.60 + 5.25 = 14.85 square inches to $\frac{4}{29}(32\,x - x^2)$; *i.e.*,

$$14.85 = \tfrac{4}{29}(32\,x - x^2)$$

Whence

$$x^2 - 32\,x + 107.66 = 0$$

i.e., $x = 16 \pm \sqrt{256 - 107.66} = 16 \pm \sqrt{148.34}$

$= 16 \pm 12.18 = 3.82'$ or $28.18'$

These are distances which, measured from *one* end of the effective span, give the two points at which the second flange plate begins and ends; it is, therefore, 24.36 feet long nett.

From the above an expression can be deduced which is general,—viz.,

$$x = \frac{l}{2} \pm \sqrt{\left(\frac{l}{2}\right)^2 - \frac{\frac{\square_x''}{w}}{2\,p_c h}}$$

$$= \frac{l}{2} \pm \sqrt{\left(\frac{l}{2}\right)^2 - \frac{2\,p_c h\,\square_x''}{w}}$$

where x is the distance in feet from centre of end supports to the point where it is necessary to *add* another flange plate, and $\square_x''$ is the sectional area *just* at the point x; w is the load per linear foot of girder; p_c is the unit stress in compression; h is the height in feet.

The foregoing is for the compression flange, and p_c is the compressive unit; and hence $\square_x''$ is the gross sectional area *at* the point x.

To adapt the expression to the tension flange, change p_c to f_t, and consider $\square_x''$ as the *nett* sectional area at the point x,—*i.e.*, for tension flange,

$$x = \frac{l}{2} \pm \sqrt{\left(\frac{l}{2}\right)^2 - \frac{2\,f_t h\,\square_x''}{w}}$$

To continue with upper flange. For the point where it is necessary to begin the third flange plate. The area of the two flange angles and the first and second flange plates is $9.60 + 5.25 + 5.25 = 20.10\,\square''$; *i.e.*, $\square_x'' = 20.10$, and

$$\frac{2\,p_c h}{w} = \frac{2 \times 4.35 \times 3\frac{1}{3}}{4.0} = \frac{29}{4}$$

Then
$$x = 16 \pm \sqrt{(16)^2 - \frac{29 \times 20.10}{4}}$$
$$= 16 \pm \sqrt{256 - 145.72}$$
$$= 16 \pm \sqrt{110.28} = 16 \pm 10.50 = 5.5' \text{ or } 26.50'$$

whence third plate is $26.5 - 5.5 = 21.0'$ long nett.

To find the length of the *fourth* flange plate. The area of the two flange angles and the first, second, third flange plates is $9.60 + 3 \times 5.25 = 25.35 \square''$; *i.e.*, $\square_x'' = 25.35$; whence

$$x = 16 \pm \sqrt{(16)^2 - \frac{29}{4}(25.35)}$$
$$= 16 \pm \sqrt{256 - 183.78}$$
$$= 16 \pm \sqrt{72.22} = 16 \pm 8.50 = 7.50' \text{ or } 24.50'$$

whence fourth flange plate is $24.50 - 7.50 = 17.0'$ long nett.

To find the length of the *fifth* or last flange plate. The area of the two flange angles and the first four flange plates $= 9.60 + 4 \times 5.25 = 30.60 \square''$; *i.e.*, $\square_x'' = 30.60$; whence

$$x = 16 \pm \sqrt{(16)^2 - \frac{29}{4}(30.60)}$$
$$= 16 \pm \sqrt{256 - 221.84}$$
$$= 16 \pm \sqrt{34.16} = 16 \pm 5.84 = 10.16' \text{ or } 21.84'$$

whence *fifth* flange plate is $21.84 - 10.16 = 11.68'$ long nett.

Conclusion:

First flange plate, $14 \times \frac{3}{8}$; full length of girder.

Second flange plate, $14 \times \frac{3}{8}$, $24.36'$ long nett, make $25\frac{1}{2}'$ long.

Third flange plate, $14 \times \frac{3}{8}$, $21.00'$ long nett, make $22\frac{1}{2}'$ long.

Fourth flange plate, $14 \times \frac{3}{8}$, $17.00'$ long nett, make $18\frac{1}{2}'$ long.

Fifth flange plate, $14 \times \frac{3}{8}$, $11.68'$ long nett, make $13\frac{1}{4}'$ long.

The above lengths are just *about* the proper lengths; the

actual "bill" length can be determined when we fix on the pitch of the rivets in each panel.

Another way to determine the lengths of flange plates is as follows:

The centre section required in upper flange is

$$\square_c'' = \frac{wl^2}{8\ p_c h}$$

Transposing,

$$\frac{l^2}{8} = \frac{p_c h\ \square_c''}{w}$$

i.e.,

$$\left(\frac{l}{2}\right)^2 = \frac{2\ p_c h\ \square_c''}{w}$$

This is the equation of a parabola, in which we may consider l and $\square''$ as variables, and calling $\frac{l}{2} = y$,

$$y^2 = \frac{2\ p_c h}{w}\ \square''$$

i.e.,

$$y = \sqrt{\frac{2\ p_c h}{w}}\ \sqrt{\square''}$$

where y represents the distance from the centre of span to point corresponding to $\square''$. See diagram, page 194.

Similarly, if we are considering the lower or tension flange,

$$y = \sqrt{\frac{2\ f_t h}{w}}\ \sqrt{\square''}$$

To illustrate, as this is an inverse method.

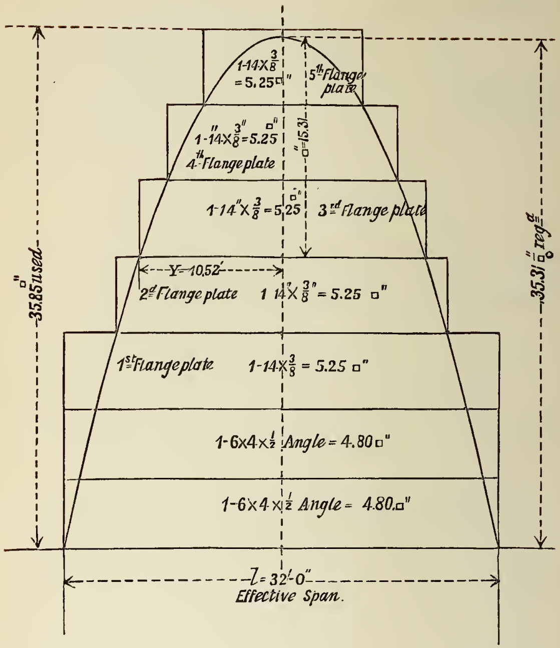
The $\square_c'' = 35.31\ \square''$. Now, 2 angles and 4 flange plates $= 9.60 + 4 \times 5.25 = 30.60\ \square''$; whence difference $= 35.31 - 30.60 = 4.81$.

Then

$$y = \sqrt{\frac{2\ p_c h}{w}}\ \sqrt{x} = \sqrt{\frac{2 \times 4.35 \times 3\frac{1}{3}}{4.0}}\ \sqrt{x}$$

$$= \sqrt{\frac{29}{4}}\ \sqrt{x} = 2.69\ \sqrt{x}$$

but x for *fifth* flange plate, or *first* plate on top, $= 4.81\ \square''$.



Lengths of flange plates should exceed the above nett lengths by about two rivet pitches at each end. Flange angles and first flange plates should extend full length of the girder.

Whence

$$y = 2.69 \sqrt{4.81} = 2.69 \times 2.19 = 5.89'$$

i.e., half length of *top* plate $= 5.89'$; whence *full* length of *top* flange plate $= 11.98$ feet nett.

Now, for the *fourth* flange plate, or the second plate from top, $\square'' = 4.81 + 5.25 = 10.06 \square''$, or, as we saw before, $35.31 - 30.06 = 10.06 \square''$; then

$$y = 2.69 \sqrt{10.06} = 2.69 \times 3.17 = 8.53'$$

i.e., *full* length of *fourth* flange plate $= 2 \times 8.53 = 17.06$ feet nett.

For the *third* flange plate, or third plate from top, $\square'' = 10.06 + 5.25 = 15.31$.

$$y = 2.69 \sqrt{15.31} = 2.69 \times 3.91 = 10.52'$$

or *full* length of *third* flange plate $= 2 \times 10.52 = 21.04$ feet nett.

For the *second* flange plate, or fourth plate from top, $\square'' = 15.31 + 5.25 = 20.35 \square''$; then

$$y = 2.69 \sqrt{20.35} = 2.69 \times 4.51 = 12.13'$$

or full length of second flange plate $= 2 \times 12.13 = 24.26$ feet; and the first flange *could* stop at

$$y = 2.69 \times \sqrt{25.60} = 2.69 \times 5.06 = 13.61$$

or full length of first flange plate $= 2 \times 13.61 = 27.22'$; but we will continue this plate from end to end of girder.

Now, for lower flange plates, use the expression

$$y = \sqrt{\frac{2 f_t h}{w}} \sqrt{\square''}$$

The *nett* sectional area required at centre of span is 25.60 square inches, and from plates used we have the following values of square inches,—viz.,

For fourth flange plate, $14 \times \frac{3}{8}$, *nett* □″ = 4.5 □″; *i.e.*, lowest plate.

For third flange plate, $2 - 14 \times \frac{3}{8}$, *nett* □″ = 9.0 □″.

For second flange plate, $3 - 14 \times \frac{3}{8}$, *nett* □″ = 13.5 □″.

For first flange plate, $4 - 14 \times \frac{3}{8}$, *nett* □″ = 18.0 □″; *i.e.*, plate next flange angles.

And

$$\sqrt{\frac{2\,f_t h}{w}} = \sqrt{\frac{2 \times 6.0 \times 3\frac{1}{3}}{4.0}} = \sqrt{10} = 3.16$$

Then general expression becomes $y = 3.16 \sqrt{\square''}$.

For *fourth* flange plate, □″ = 4.5 nett; then

$$y = 3.16 \sqrt{4.5} = 3.16 \times 2.12 = 6.70'$$

i.e., half length = 6.70′, whence full length = 2 × 6.70 = 13.40′ long nett.

For third flange plate, □″ = 9.0 □″ nett; then

$$y = 3.16 \sqrt{9.0} = 3.16 \times 3.0 = 9.48'$$

i.e., half length = 9.48′, or full length = 2 × 9.48 = 18.96′ long nett.

For second flange plate, □″ = 13.5 □″ nett; then

$$y = 3.16 \sqrt{13.5} = 3.16 \times 3.67 = 11.60'$$

i.e., half length = 11.60′, whence full length = 2 × 11.60 = 23.20′ long nett.

For first flange plate, □″ = 18.00 □″ nett; then

$$y = 3.16 \sqrt{18} = 3.16 \times 4.24 = 13.40'$$

i.e., half length = 13.40′, or full length = 2 × 13.40 = 26.80′ long nett. But the first flange plate, being next to the flange angles, it should extend the full length of girder.

Conclusion:

First flange plate, $14 \times \frac{3}{8}$; required length = 26.80′ nett; make full length.

Second flange plate, $14 \times \frac{3}{8}$; required length = 23.20′ nett; make 25.0′.

Third flange plate, $14 \times \frac{3}{8}$; required length $= 18.96'$ nett; make $20.5'$.

Fourth flange plate, $14 \times \frac{3}{8}$; required length $= 13.40'$ nett; make $15.0'$.

The above lengths are *about* the proper lengths to be used; the actual "bill" lengths can be determined when the pitch of rivets in each panel is known, and a drawing is made.

To determine the thickness of the web sheet in each panel, we will need the *shear* at *centre* of each panel.

To determine the diameter and pitch of the rivets in each panel, we will find the shears at each *panel point*, and, determining the diameter and pitch of rivets at these points, will continue such pitch to next panel point towards the centre of span. In other words, the pitch of the rivets in any panel will be determined by the shear at the end of such panel towards abutment.

The general expression for the shear at any point is

$$F_x = w\left\{\frac{l}{2} - x\right\} = 4.0\left\{16 - x\right\}$$

Then shear at

Supports, $x = 0$; whence $F_0 = 4 \times 16 = 64.00$ tons.

Centre of first panel, $x = 2.0'$; whence $F_2 = 4 \times 14 = 56.00$ tons.

First panel point, $x = 4.0'$; whence $F_4 = 4 \times 12 = 48.00$ tons.

Centre of second panel, $x = 6.0'$; whence $F_6 = 4 \times 10 = 40.00$ tons.

Second panel point, $x = 8.0'$; whence $F_8 = 4 \times 8 = 32.00$ tons.

Centre of third panel, $x = 10.0'$; whence $F_{10} = 4 \times 6 = 24.00$ tons.

Third panel point, $x = 12.0'$; whence $F_{12} = 4 \times 4 = 16.00$ tons.

Centre of fourth panel, $x = 14.0'$; whence $F_{14} = 4 \times 2 = 8.00$ tons.

Fourth panel point, or centre of span, $x = 16.0'$; whence $F_c = 4 \times 0 = 0$ tons.

To resist the crippling of the web sheet, the unit stress should be determined from

$$p_{ps} = \frac{5.0 \text{ tons}}{1 + \frac{1}{3000}\left(\frac{h}{t}\right)^2}$$

where h may be taken as the distance in the *clear* between the flange angles, and which here $= 40'' - 2 \times 4'' = 32''$; and t is the thickness of the web in inches.

We will use no web sheet less than $\frac{3}{8}''$ thick; whence for

$t = \frac{3}{8}''$; $\frac{h}{t} = 85$; then $p_{ps} = \frac{5.0}{3.41} = 1.47$ tons per sq. in.

$t = \frac{7}{16}''$; $\frac{h}{t} = 73$; then $p_{ps} = \frac{5.0}{2.78} = 1.80$ tons per sq. in.

$t = \frac{1}{2}''$; $\frac{h}{t} = 64$; then $p_{ps} = \frac{5.0}{2.37} = 2.11$ tons per sq. in.

$t = \frac{9}{16}''$; $\frac{h}{t} = 57$; then $p_{ps} = \frac{5.0}{2.08} = 2.40$ tons per sq. in.

$t = \frac{5}{8}''$; $\frac{h}{t} = 51$; then $p_{ps} = \frac{5.0}{1.87} = 2.69$ tons per sq. in.

Now, at any panel centre, we should have $p_{ps}ht = F_x$; whence

$$p_{ps} \cdot t = \frac{F_x}{h}$$

where t and h are in inches. If we take t in *inches* and h in *feet*, the above becomes

$$12 \cdot p_{ps} \cdot t = \frac{F_x}{h'}$$

i.e., $12 \cdot p_{ps} \cdot t =$ shear at centre of panel divided by the height in *feet* = shear per foot at centre of panel.

Now

$12 \cdot p_{ps} \cdot t$ for $\frac{3}{8}''$ web $= 12 \times 1.47 \times \frac{3}{8} = 6.62$ tons per foot;

and

$12 \cdot p_{ps} \cdot t$ for $\frac{7}{16}''$ web $= 12 \times 1.80 \times \frac{7}{16} = 9.45$ tons per foot;

and

$12 \cdot p_{ps} \cdot t$ for $\frac{1}{2}''$ web $= 12 \times 2.11 \times \frac{1}{2} = 12.66$ tons per foot; and

$12 \cdot p_{ps} \cdot t$ for $\frac{9}{16}''$ web $= 12 \times 2.40 \times \frac{9}{16} = 16.2$ tons per foot; and

$12 \cdot p_{ps} \cdot t$ for $\frac{5}{8}''$ web $= 12 \times 2.69 \times \frac{5}{8} = 20.18$ tons per foot.

And at centre of

First panel, $F_x \div h = 56.00 \div 3\frac{1}{3}' = 16.80$ tons per foot.
Second panel, $F_x \div h = 40.00 \div 3\frac{1}{3}' = 12.00$ tons per foot.
Third panel, $F_x \div h = 24.00 \div 3\frac{1}{3}' = 7.20$ tons per foot.
Fourth panel, $F_x \div h = 8.00 \div 3\frac{1}{3}' = 2.40$ tons per foot.

Now, remembering that in any case $12 \cdot p_{ps} \cdot t = \frac{F_x}{h'}$, we can use, by inspection of above,

In first panel, a $\frac{9}{16}''$ web.
In second panel, a $\frac{1}{2}''$ web.
In third panel, a $\frac{3}{8}''$ web.
In fourth panel, a $\frac{3}{8}''$ web.

For in first panel we require a resistance of 16.80 tons per foot, and by using a $\frac{9}{16}''$ web, we have 16.20 tons per foot. In second panel we require a resistance of 12.00 tons per foot, and by using a $\frac{1}{2}''$ web, we have 12.66 tons per foot. In third panel we require a resistance of 7.20 tons per foot, and by using a $\frac{3}{8}''$ web, we have 6.62 tons per foot, which is close enough. In fourth panel we require a resistance of 2.40 tons per foot, and using no web less than $\frac{3}{8}''$ thick, we have 6.62 tons per foot.

It is desirable to make as few joints in the web as possible, even at the expense of weight of iron; so we will use a $\frac{9}{16}''$ web, extending from end of girder to the second panel point, and a $\frac{3}{8}''$ web between the second panel points, from each end. There will then be but two joints in web, and at points where the shear $= 32.00$ tons; for at the distance $x = 8$, $F_8 = 4 \times 8 = 32.00$ tons. The splice will be proportioned after we have determined the diameter and pitch of the rivets.

To determine the diameter and pitch of the rivets. The number of rivets per foot required at any point distant x

from the abutment = shear per foot at the point divided by the allowable stress on the rivet,—*i.e.*,

$$\text{n per foot} = \frac{\frac{F_x}{h'}}{a}$$

Now shear *per foot* at the point

x = 0, or end of girder = 64 tons ÷ $3\frac{1}{3}'$ = 19.20 tons per foot.

x = 4.0′, or first panel point = 48 tons ÷ $3\frac{1}{3}'$ = 14.40 tons per foot.

x = 8.0′, or second panel point = 32 tons ÷ $3\frac{1}{3}'$ = 9.60 tons per foot.

x = 12.0′, or third panel point = 16 tons ÷ $3\frac{1}{3}'$ = 4.80 tons per foot.

x = 16.0′, or fourth panel point = 0 ÷ $3\frac{1}{3}'$ = 0.

And using $\frac{7}{8}''$ rivets; a $\frac{7}{8}''$ rivet in *double* shear between the flange angles at 4.5 tons per square inch = 2 × 2.70 = 5.40 tons. (See Table of Shearing Value of Rivets.) And a $\frac{7}{8}''$ rivet in a $\frac{9}{16}''$ web, with a bearing unit of 9.0 tons per square inch, has a value of 4.43 tons. (See Table of Bearing Value of Rivets.) Also, a $\frac{7}{8}''$ in a $\frac{3}{8}''$ web has a bearing value of 2.95 tons. Whence the bearing values in both cases of $\frac{9}{16}''$ and $\frac{3}{8}''$ web is less than the shearing values, and we see the allowable stress *a* in the panels which have a $\frac{9}{16}''$ web is 4.43 tons, and in the panels which have a $\frac{3}{8}''$ web is 2.95 tons; then

In *first* panel,

$$\text{n per foot} = \frac{19.20}{4.43} = 4.33;$$

i.e., we require $4\frac{1}{3}$ rivets $\frac{7}{8}''$ diameter per foot; whence

$$pitch = \frac{12''}{4\frac{1}{3}} = 2.77'', \text{ which we can call } 2\frac{3}{4}''.$$

In *second* panel, having a $\frac{9}{16}''$ web,

$$\text{n per foot} = \frac{14.40}{4.43} = 3.25 = 3.69''\text{ pitch say, } 3\frac{1}{2}''\text{ pitch.}$$

In *third* panel, having a $\frac{3}{8}''$ web,

$$n \text{ per foot} = \frac{9.60}{2.95} = 3.25 = 3.69'' \text{ pitch, say } 3\tfrac{1}{2}'' \text{ pitch.}$$

In *fourth* panel, having a $\frac{3}{8}''$ web,

$$n \text{ per foot} = \frac{4.8}{2.95} = 1.625 = 7.38'' \text{ pitch, say } 6'' \text{ pitch,}$$

because the flange plates being $\frac{3}{8}''$ thick, the pitch in them to angles (the rivets "breaking joint" with those in flange angles to web) is limited to $16 \times \frac{3}{8} = 6''$.

Whence we have

In first panel, web $\frac{9}{16}''$; pitch $= 2\frac{3}{4}''$ in flange angles to web.

In second panel, web $\frac{9}{16}''$; pitch $= 3\frac{1}{2}''$ in flange angles to web.

In third panel, web $\frac{3}{8}''$; pitch $= 3\frac{1}{2}''$ in flange angles to web.

In fourth panel, web $\frac{3}{8}''$; pitch $= 6''$ in flange angles to web.

And the pitch in flange plates to flange angles will be the same in each panel as above, and "break joint" with them. But the flange plates being $14''$ wide, and the horizontal leg of the flange angles being $6''$ wide each, there should be two lines of rivets in each horizontal leg,—*i.e.*, four lines of rivets in the flange plates; whence the pitch of rivets on *each* line should be *double* the pitch of rivets in the vertical leg of angle to web in the panel under consideration, and so arranged that no more than two holes are deductive in *each* angle iron, for, in proportioning the tension flange, a deduction for two holes is made in *each* angle iron.

Now for the joint between the $\frac{9}{16}''$ and $\frac{3}{8}''$ webs, at the point 8.0 feet from abutment. The shear at this point is $F_8 = 4\,(16 - 8) = 32.0$ tons. The shear per foot $= \frac{32}{3\frac{1}{3}} =$ 9.60 tons per foot. The shearing unit on plate $f_{ps} = 5.00$ tons; whence we need $\frac{32}{5} = 6.4$ square inches nett area in a vertical section of the splices. These splices are $40 - 2 \times 4 = 32''$ long in height, and one on each side of web.

The nett sectional area of these splices is

$$2\left\{32 - \text{number of rivet holes in the height of } 32''\right\}t'',$$

where t is the thickness of each vertical splice plate. Now, the number of rivets required on each side of the vertical joint in the vertical dimension of splice is $= 32.00$ tons $\div$ allowable stress on the rivet $= 32.00 \div 2.95 = 11.8$, say 12; the allowable stress being for bearing in $\frac{3}{8}''$ web, that being less than the shearing value of a $\frac{7}{8}''$ rivet in double shear.

$$\text{Then pitch required vertically} = \frac{30''}{12 \text{ rivets}} = 2\tfrac{2}{3}'', \text{ say } 2\tfrac{5}{8}'';$$

or, as plate is 32″ long, and extreme rivet holes should be $1\frac{1}{2}''$ from ends, we have a height of $32 - 2 \times 1\frac{1}{2} = 29''$; and having 12 rivets, there are 11 spaces; whence spacing or pitch $= \frac{29}{11} = 2.63''$, if evenly pitched $=$ say $2\frac{5}{8}''$.

The stiffeners may be made of $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angle irons, two at each panel point, and on opposite sides of web. At the intermediate panel points, where no splice occurs, the "fillers" between vertical stiffening angles and web sheet are $3\frac{1}{2}'' \times \frac{1}{2}''$, 32″ long in height, $\frac{1}{2}''$ being same as thickness of flange angles. At splice in web, the splices are $7'' \times \frac{1}{2}''$, 32″ long in height, and on them, one on each side of girder, is a $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angle iron stiffener, as at other points. There are two vertical lines of rivets, 4″ apart horizontally, the vertical pitch being $2\frac{5}{8}''$, as determined above. At ends of girder over supports there should be two pairs of stiffeners, as per sketch, the distance apart of which is governed by thickness of wall on which the girders rest. For girders bearing such heavy loads as this, the "filler" plate should extend from back to back of the pair of stiffeners. Thus, if bearing were 18″ wide, the stiffeners back to back would be 18″; and the "fillers" could then be $18'' \times \frac{1}{2}''$, 32″ high, one on each side of web. The distance apart, centre to centre of stiffeners, would then be $15'' =$ say five spaces, at 3″ each; and the vertical pitch in the stiffeners could be 3″ likewise.

If there were but *one* pair of stiffeners over end support, and but *one* line of rivets vertically, the pitch should be the

same as determined for first panel,—viz., $2\frac{3}{4}''$. Taking the girder $33'.6''$ long from end to end, the approximate bill and weight of this girder is as follows, bearing in mind that the web sheets should be $\frac{1}{2}''$ less in height than the distance out to out of angles, to allow for inequality of sheared edges of web, and the lengths of the web plates $\frac{1}{2}''$ less in length, for a like reason:

		Lbs.
Upper flange. Two $6'' \times 4'' \times \frac{1}{2}''$ angles, 48 pounds per yard, $33'.6''$ long		1070
One plate, $14 \times \frac{3}{8}$, $33'.6''$ long One plate, $14 \times \frac{3}{8}$, $25'.6''$ long One plate, $14 \times \frac{3}{8}$, $22'.6''$ long One plate, $14 \times \frac{3}{8}$, $18'.6''$ long One plate, $14 \times \frac{3}{8}$, $13'.3''$ long	$113\frac{1}{4}$ linear feet.	2000
Lower flange. Two $6'' \times 4'' \times \frac{1}{2}''$, angles, 48 pounds per yard, $33'.6''$ long		1070
One plate, $14 \times \frac{3}{8}$, $33'.6''$ long One plate, $14 \times \frac{3}{8}$, $25'.0''$ long One plate, $14 \times \frac{3}{8}$, $20'.6''$ long One plate, $14 \times \frac{3}{8}$, $15'.0''$ long	94 linear feet	1660
Rivet heads. 1st, in flange plates to angles.		
16 lines $\frac{7}{8}''$ rivet heads, $5\frac{1}{2}''$ pitch, $9\frac{1}{2}'$ long 16 lines $\frac{7}{8}''$ rivet heads, $7''$ pitch, $16'$ long 16 lines $\frac{7}{8}''$ rivet heads, $12''$ pitch, $8'$ long		200
2d, in flange angles to web.		
4 lines $\frac{7}{8}''$ rivet heads, $2\frac{3}{4}''$ pitch, $9\frac{1}{2}'$ long 4 lines $\frac{7}{8}''$ rivet heads, $3\frac{1}{2}''$ pitch, $16'$ long 4 lines $\frac{7}{8}''$ rivet heads, $6''$ pitch, $8'$ long		100
Two ends over supports.		
Eight $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles, 24.9 pounds per yard, $3'.3''$		215
Four plates, $18'' \times \frac{1}{2}''$, $2'.8''$ long		325
Twenty lines $\frac{7}{8}''$ rivet heads, $3''$ pitch, $3\frac{1}{3}'$		55

	Lbs.
Four stiffeners.	
Eight $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles, 24.9 pounds per yard, $3'.3''$	215
Eight bars, $3\frac{1}{2}'' \times \frac{1}{2}''$, $2'.8''$ long	125
Eight lines $\frac{7}{8}''$ rivet heads, $3''$ pitch, $3\frac{1}{3}'$	25
Two splices.	
Four $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles, 24.9 pounds per yard, $3'.3''$	105
Four flats, $7'' \times \frac{1}{2}''$, $2'.8''$ long	125
Eight lines $\frac{7}{8}''$ rivet heads, $2\frac{5}{8}''$ pitch, $3\frac{1}{3}'$	25
Three web sheets.	
Two plates, $39\frac{1}{2}'' \times \frac{9}{16}''$, $8'.8\frac{1}{2}''$ = 1285 pounds One plate, $39\frac{1}{2}'' \times \frac{3}{8}''$, $15'.11\frac{1}{2}''$ = 790 pounds	2075
	9390

	Lbs.
Flanges	6100
Ends	595
Stiffeners	365
Splices	255
Web sheets	2075
	9390

The bearing pressure on brick walls should not exceed 8.0 tons per square foot, and if the above girders rest on brickwork, the bearing area needed is $\frac{64}{8} = 8.0$ square feet $= 1152$ square inches. This would require a stone 5.0 feet long if the wall be $18''$ wide, for $60 \times 18 = 1080$ square inches.

For such heavy girders there should be a pilaster built under the ends, and, covering it and the wall, should be set a stone block not less than $5''$ thick.

On stone, the bearing should not exceed 300 pounds per square inch; whence area of plate required under ends of the girder, between it and the stone, is 64 tons $\div$ 0.15 tons $= 427$ square inches, say $18''$ wide, $24''$ long, which equals 432 square inches. Its thickness should be, for such a heavy girder, $1''$.

EXAMPLE II.

DOUBLE-WEBBED PLATE GIRDER;

i.e., a Box Girder.

Taking the same effective span, height, and load as in Example I., we have

$$l = 32'.0''.$$
$$h = 3'.4'' = 40''.$$
$$w = 4.0 \text{ tons.}$$
$$M_c = 512 \text{ foot-tons} = 6144 \text{ inch-tons.}$$

As the width of a single top flange plate may not exceed thirty times the distance centre to centre of rivets across the plate, allowing 2″ from centre of each rivet hole to edge of plate, for a

$\frac{3}{8}''$ plate, maximum width $= 30 \times \frac{3}{8} + 2 \times 2'' = 15.25''$.
$\frac{1}{2}''$ plate, maximum width $= 30 \times \frac{1}{2} + 2 \times 2'' = 19.00''$.
$\frac{5}{8}''$ plate, maximum width $= 30 \times \frac{5}{8} + 2 \times 2'' = 22.75''$.
$\frac{3}{4}''$ plate, maximum width $= 30 \times \frac{3}{4} + 2 \times 2'' = 26.50''$.

If, then, we use a 20″ plate, its distance across centres of rivet holes will be about 16″, and its thickness must be $\frac{16}{30}''$ $= 0.53''$; or, we might say, the minimum thickness of first flange plate $= \frac{1}{2}''$. The ratio of length of girder to width of flange $= 32 \div 1\frac{2}{3} = 19$,

$$p_c = \frac{5.0}{1 + \frac{1}{5000}\left(19\right)^2} = \frac{5.0}{1.120} = 4.47 \text{ tons per square inch,}$$

which is the maximum permissible stress on upper flange.

Then $\square_c''$ required at centre of upper flange

$$= \frac{M_c}{p_c h''} = \frac{6144}{4.47 \times 40} = \frac{6144}{178.8} = 34.36 \square'' \text{ gross,}$$

and $\square_t''$ required at centre of lower flange

$$= \frac{M_c}{f_t h''} = \frac{6144}{6 \times 40} = \frac{6144}{240} = 25.60 \square'' \text{ nett.}$$

For compression flange—*i.e.*, the top flange—use

	Sq. in.
2—$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$ angles, 33.6 lbs. per yard =	6.72
1 first top plate, $20 \times \frac{1}{2} =$	10.00
1 second top plate, $20 \times \frac{1}{2} =$	10.00
1 third top plate, $20 \times \frac{3}{8} =$	7.50
Total gross section used in upper flange =	34.22

For tension flange—*i.e.*, the lower flange—use

		Sq. in., nett.
2—$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$ angles, 33.6 lbs. per yard =	6.72	
Deduct *two* holes, $1'' \times \frac{1}{2}'' =$	1.00 =	5.72
3 flange plates, $20 \times \frac{3}{8} =$	22.50	
Deduct 3 (two holes, $1'' \times \frac{3}{8}''$) =	2.25 =	20.25
Total nett section used in lower flange =		25.97

To determine lengths of upper flange plates, we have

$$y = \sqrt{\frac{2\ p_c h'}{w}}\sqrt{\square''} = \sqrt{\frac{2 \times 4.47 \times 3\frac{1}{3}}{4.0}}\sqrt{\square''}$$

$$\sqrt{7.45}\sqrt{\square''} = 2.73\sqrt{\square''}$$

For *third* top flange plate, $\square'' = 34.36 - 26.72 = 7.64$ $\square''$, then $y = 2.73\sqrt{7.64} = 2.73 \times 2.76 = 7.53'$; whence full nett length $= 2 \times 7.53 = 15.06$ feet.

For *second* top flange plate, $\square'' = 34.36 - 16.72 = 17.64$ $\square''$, then $y = 2.73\sqrt{17.64} = 2.73 \times 4.20 = 11.47'$; whence full nett length $= 2 \times 11.47 = 22.94$ feet.

For *first* top flange plate, $\square'' = 34.36 - 6.72 = 27.64$ $\square''$, then $y = 2.73\sqrt{27.64} = 2.73 \times 5.26 = 14.36'$; whence full nett length $= 2 \times 14.36 = 28.72$ feet. This plate, however, must extend the full length of girder from end to end.

For lengths of lower flange plates, we have

$$y = \sqrt{\frac{2\ f_t h'}{w}}\sqrt{\square''} = \sqrt{\frac{2 \times 6 \times 3\frac{1}{3}}{4}}\sqrt{\square''}$$

$$= \sqrt{10}\sqrt{\square''} = 3.16\sqrt{\square''}$$

For *third* flange plate, $\square'' = 25.60 - 19.22 = 6.38 \square''$, then $y = 3.16 \sqrt{6.38} = 3.16 \times 2.53 = 7.99'$; whence full nett length $= 2 \times 7.99 = 15.98$ feet.

For *second* flange plate, $\square'' = 25.60 - 12.47 = 13.13 \square''$, then $y = 3.16 \sqrt{13.13} = 3.16 \times 3.62 = 11.44'$; whence full nett length $= 2 \times 11.44 = 22.88$ feet.

For *first* flange plate, $\square'' = 25.60 - 5.72 = 19.88 \square''$, then $y = 3.16 \sqrt{19.88} = 3.16 \times 4.46 = 14.09'$; whence full nett length $= 2 \times 14.09 = 28.18$ feet. This plate, however, should extend full length of girder from end to end.

Conclusion:

Upper flange.

First flange plate, $20 \times \frac{1}{2}$; required length $= 28.72'$; make full length.

Second flange plate, $20 \times \frac{1}{2}$; required length $= 22.94'$; make $24'.6''$.

Third flange plate, $20 \times \frac{3}{8}$; required length $= 15.06'$; make $16'.6''$.

Lower flange.

First flange plate, $20 \times \frac{3}{8}$; required length, $28.18'$; make full length.

Second flange plate, $20 \times \frac{3}{8}$; required length, $22.88'$; make $24'.6''$.

Third flange plate, $20 \times \frac{3}{8}$; required length, $15.98'$; make $17'.6''$.

The shears per foot on *each* web sheet at centre of panels are

In first panel, $16.80 \div 2 = 8.40$ tons per foot.
In second panel, $12.00 \div 2 = 6.00$ tons per foot.
In third panel, $7.20 \div 2 = 3.60$ tons per foot.
In fourth panel, $2.40 \div 2 = 1.20$ tons per foot.

The permissible stress per square inch on the web sheets is determined by

$$p_{ps} = \frac{5.0 \text{ tons.}}{1 + \frac{1}{3000}\left(\frac{h}{t}\right)^2}$$

where $h = 40 - 2 \times 3\frac{1}{2} = 33''$ and, considering $\frac{3}{8}''$ as the minimum thickness to be used, we get for

$$t = \tfrac{3}{8}''; \quad \frac{h}{t} = 88; \text{ then } p_{ps} = \frac{5.00}{3.58} = 1.40 \text{ tons per sq. in.}$$

$$t = \tfrac{7}{16}''; \quad \frac{h}{t} = 76; \text{ then } p_{ps} = \frac{5.00}{2.92} = 1.71 \text{ tons per sq. in.}$$

$$t = \tfrac{1}{2}''; \quad \frac{h}{t} = 66; \text{ then } p_{ps} = \frac{5.00}{2.45} = 2.04 \text{ tons per sq. in.}$$

and

$12\, p_{ps}t$ for $\frac{3}{8}''$ web $= 12 \times 1.40 \times \frac{3}{8} = 6.30$ tons per foot.

and

$12\, p_{ps}t$ for $\frac{7}{16}''$ web $= 12 \times 1.71 \times \frac{7}{16} = 9.00$ tons per foot.

and

$12\, p_{ps}t$ for $\frac{1}{2}''$ web $= 12 \times 2.04 \times \frac{1}{2} = 12.24$ tons per foot.

Now, remembering that $12\, p_{ps}t$ should equal or exceed $\frac{F_x}{h'}$, we can, by inspection of above, proportion the web sheets.

In first panel, need 8.40 tons per foot resistance. A $\frac{7}{16}''$ web has 9.00 tons; whence use $\frac{7}{16}''$ web in first panel. In second panel, need 6.00 tons per foot resistance. A $\frac{3}{8}''$ web has 6.30 tons; whence can use $\frac{3}{8}''$ web in second panel. And as no web sheet may be less than $\frac{3}{8}''$, all other web sheets are $\frac{3}{8}''$.

We shall splice the web at the second panel point, so use a $\frac{7}{16}''$ plate for each web, from 0 to $8'$ from centre of end supports, and a $\frac{3}{8}''$ web between the second panel points from each end. There will then be but two splices in each web, and at a point where the shear is 16.00 tons on each web, or a total of 32.0 tons per girder.

To determine the rivet diameter and pitch,

$$n \text{ per foot} = \frac{\dfrac{F_x}{h}}{a}$$

Now shear per foot on *each* web at the point

$x = 0$, or end of girder $= 19.20 \div 2 = 9.60$ tons per foot.
$x = 4'$, or first panel point $= 14.40 \div 2 = 7.20$ tons per foot.
$x = 8'$, or second panel point $= 9.60 \div 2 = 4.80$ tons per foot.
$x = 12'$, or third panel point $= 4.80 \div 2 = 2.40$ tons per foot.
$x = 16'$, or fourth panel point $= 0 = 0$ tons per foot.

Using $\frac{7}{8}''$ rivets; a $\frac{7}{8}''$ rivet in single shear in connexion of flange angle to web at 4.5 tons per square inch $= 2.70$ tons. And a $\frac{7}{8}''$ rivet in a $\frac{7}{16}''$ web at 9.0 tons per square inch, has a bearing value of 3.44 tons; also, a $\frac{7}{8}''$ rivet in a $\frac{3}{8}$ plate has, at 9.0 tons per square inch, a bearing value of 2.95 tons. Whence, the shearing value being the less in each case, the allowable stress a on the rivets in *all* the panels is 2.70 tons.

In the first panel we have

$$n \text{ per foot} = \frac{9.60}{2.70} = 3.56$$

which equals a pitch of

$$\frac{12}{3.56} = 3.37'', \text{ say } 3''$$

In the second panel we have

$$n \text{ per foot} = \frac{7.20}{2.70} = 2\tfrac{2}{3} = 4\tfrac{1}{2}'' \text{ pitch.}$$

In the third panel we have

$$n \text{ per foot} = \frac{4.80}{2.70} = 1.78 = 6\tfrac{3}{4}'', \text{ say use } 6'' \text{ pitch.}$$

Result in *each* web.

First panel, web $\frac{7}{16}''$, pitch $= 3''$ in flange angle to web.
Second panel, web $\frac{7}{16}''$, pitch $= 4\frac{1}{2}''$ in flange angle to web.
Third panel, web $\frac{3}{8}''$, pitch $= 6''$ in flange angle to web.
Fourth panel, web $\frac{3}{8}''$, pitch $= 6''$ in flange angle to web.

Maximum pitch in flanges $= 16 \times \frac{3}{8} = 6''$; whence no pitch greater than $6''$ throughout girder. Whence in flange plates,

Over first panel, pitch $3''$, and "breaking joint" with those in web,

Over second panel, pitch $4\frac{1}{2}''$, and "breaking joint" with those in web.

Over third panel, pitch $6''$, and "breaking joint" with those in web.

Over fourth panel, pitch $6''$, and "breaking joint" with those in web.

For the joint between the $\frac{7}{16}''$ and $\frac{3}{8}''$ web, the shear on *each* web = 16.00 tons; the allowable stress a on the $\frac{7}{8}$ rivet being due to single shear = 2.70, then number of rivets required on *each* side of the vertical joint $= \dfrac{16.0}{2.7} = 5.9$, say 6 required.

The height of the splice plate being $40 - 7'' = 33''$; then pitch required vertically $= \frac{33}{6} = 5.5''$. This we will make $4\frac{1}{2}''$, to agree with pitch in the adjoining panels. The splice plate we will make $7 \times \frac{1}{2}$, $33''$ long, two rows of rivets, $3\frac{1}{2}''$ apart horizontally, and $4\frac{1}{2}''$ vertical pitch.

All stiffeners will be $3'' \times 3'' \times \frac{3}{8}''$, and have fillers of $3'' \times \frac{1}{2}''$, $33''$ long. At the splice we will use two stiffeners, $3'' \times 3'' \times \frac{3}{8}''$ on *each* web, and set back to back.

At the end supports will use three stiffeners of $3'' \times 3'' \times \frac{3}{8}''$ angle iron on each web, and one filler plate, $18 \times \frac{1}{2}$, $33''$ long in height, and the vertical pitch in each will make $4\frac{1}{2}''$. If we used but *one* stiffener here, the pitch would have to be $3''$, the same as in first panel of flange rivets. The bearing plate will be as in Example I.,—viz., $18'' \times 24'' \times 1''$.

Taking the girder $33'.6''$ long, out to out, the approximate bill and estimated weight will be

		Lbs.
Upper flange. Two $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ angles, 33.6 pounds per yard, $33'.6''$ long		750
One plate, $20 \times \frac{1}{2}$, $33'.6''$ long	$74\frac{1}{2}$ linear feet.	2,365
One plate, $20 \times \frac{1}{2}$, $24'.6''$ long		
One plate, $20 \times \frac{3}{8}$, $16'.6''$ long		
Lower flange. Two $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ angles, 33.6 pounds per yard, $33'.6''$ long		750
One plate, $20 \times \frac{3}{8}$, $33'.6''$ long	$75\frac{1}{2}$ linear feet.	1,910
One plate, $20 \times \frac{3}{8}$, $24'.6''$ long		
One plate, $20 \times \frac{3}{8}$, $17'.6''$ long		

Rivet heads. 1st, in flange plates to angles.	
8 lines $\frac{7}{8}''$ rivet heads, 3″ pitch, $9\frac{1}{2}'$ long; 8 lines $\frac{7}{8}''$ rivet heads, $4\frac{1}{2}''$ pitch, 8′ long; 8 lines $\frac{7}{8}''$ rivet heads, 6″ pitch, 16′ long . .	165
2d, in flange angles to web.	
8 lines $\frac{7}{8}''$ rivet heads, 3″ pitch, $9\frac{1}{2}'$ long; 8 lines $\frac{7}{8}''$ rivet heads, $4\frac{1}{2}''$ pitch, 8′ long; 8 lines $\frac{7}{8}''$ rivet heads, 6″ pitch, 16′ long . .	165
Two ends over supports.	
Twelve angles, $3 \times 3 \times \frac{3}{8}''$, 21.6 pounds per yard, 3′.3″	280
Four plates 18″ $\times$ $\frac{1}{2}''$, 2′.9″ long	330
Forty lines $\frac{7}{8}''$ rivet heads, $4\frac{1}{2}''$ pitch, $3\frac{1}{3}'$ long .	75
Four stiffeners per web.	
Eight angles, $3 \times 3 \times \frac{3}{8}''$, 21.6 pounds per yard, 3′.3″	185
Eight bars, $3 \times \frac{1}{2}$, 2′.9″	110
Sixteen lines $\frac{7}{8}''$ rivet heads, $4\frac{1}{2}''$ pitch, $3\frac{1}{3}'$ long .	30
Two splices in each web.	
Eight angles, $3 \times 3 \times \frac{3}{8}''$, 21.6 pounds per yard, 3′.3″	185
Four flats, $7 \times \frac{1}{2}$, 2′.9″	130
Sixteen lines $\frac{7}{8}''$ rivet heads, $4\frac{1}{2}''$ pitch, $3\frac{1}{3}'$ long .	30
Six web sheets.	
Four plates, $39\frac{1}{2} \times \frac{7}{16}$, 8′. $8\frac{1}{2}''$ = 2040; Two plates, $39\frac{1}{2} \times \frac{3}{8}$, 15′.$11\frac{1}{2}''$ = 1580	3,620
	11,080

	Lbs.
Flanges	6,105
Ends	685
Stiffeners	325
Splices	345
Web sheets	3,620
	11,080

Whence box girder of same depth as single-webbed girder weighs 18 per cent. more. This is due principally to limiting the web sheets to a minimum thickness of $\frac{3}{8}''$.

BUCKLED PLATES.

Buckled plates are rectangular or square wrought iron or steel plates, shaped under the hammer, so as to have a slight convexity in the middle and a flat rim around the four sides, called the "fillet." They are so placed that the convex part is compressed and the flat fillet stretched; and when they are crippled, it is usually by the convex part crushing.

The plates in general use are made most frequently 3 feet square, the curvature about 2″, and the fillets about 2″. The thickness varies from $\frac{1}{4}$″ to $\frac{3}{8}$″, the $\frac{1}{4}$″ plates being amply sufficient for floors of buildings. The $\frac{3}{8}$″ plates are those used for roadway bridge floors, under a heavy road covering.

The stiffness of buckled plates is as the square of the thickness, and inversely as the curvature. According to the table of safe loads published by the inventor, Mr. Mallet, a 36″ square buckled plate has the following values for varying thicknesses:

$\frac{3}{16}$″ thickness, safe load per plate = 5,600 pounds.
$\frac{1}{4}$″ thickness, safe load per plate = 10,000 pounds.
$\frac{5}{16}$″ thickness, safe load per plate = 14,000 pounds.
$\frac{3}{8}$″ thickness, safe load per plate = 20,000 pounds.

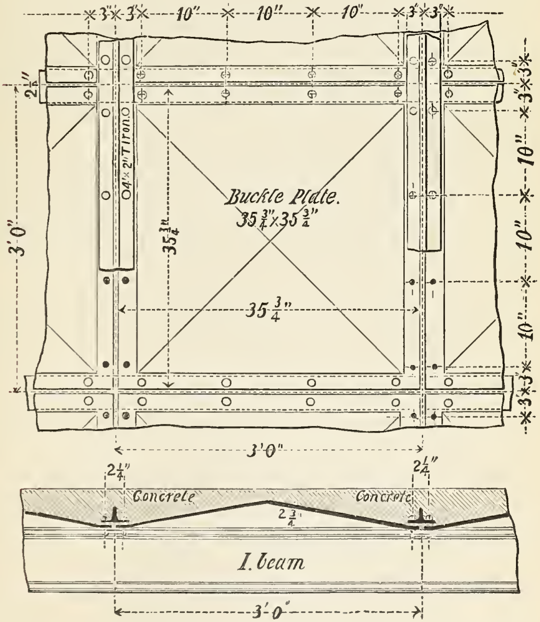
In using these plates, they generally rest on the upper flanges of beam, to which they are riveted, and the transverse joints between the buckled plates are covered by ⊥ irons, with a minimum horizontal flange of 4″. These ⊥ irons are also riveted to the fillets. An iron platform is then formed, thoroughly connected together; and on this surface is laid a concrete covering, if for building purposes. If for bridge roadway, asphalt covering is used, on which is laid the Belgian block roadway.

It is easily seen that the widths of the flanges of the beams on which the buckled plates rest should not be less than about 4″.

The actual dimensions of a buckled plate for 3′.0″ spacing of beams, showing the rivet pitch, etc., are given by the following sketch. The *rise* or convexity of this plate is rather larger than usual. A rivet spacing of 10″ is quite close enough.

In laying the plates, the transverse joints "break joint" with one another. The sketch, however, shows them in the same transverse line.

The weights of 36″ square buckled plates are as follows: $\frac{1}{8}$″ thick, 45 pounds per plate; $\frac{3}{16}$″ thick, 70 pounds per plate; $\frac{1}{4}$″ thick, 90 pounds per plate; $\frac{5}{16}$″ thick, 115 pounds per plate; $\frac{3}{8}$″ thick, 135 pounds per plate.



BUCKLED PLATE FLOORS.

A very excellent floor is made by using buckled plates *on* the floor joist, instead of brick arches *between* them.

The buckled plates are generally 3 feet square and $\frac{1}{4}''$ thick, and are riveted to the top flanges of the **I** beam joist, which are likewise spaced 3 feet apart. Over the transverse joints of the buckled plates are riveted ⊥ irons. The transverse joints should generally "break joint" with the adjacent ones. Above the buckled plates is concrete, the top surface of which should be about 1″ above the crown of the buckled plate,—that is, about 4″ above the top flanges of beams. (See sketch, page 213.) If the transverse joints of plates be in one line, the ⊥ iron may be made in one continuous length.

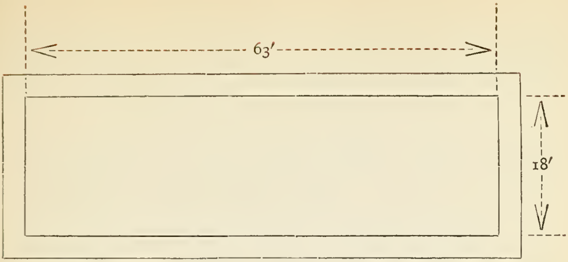
The weight of a floor of this kind, with a ceiling hung to the bottoms of the beams, will be about 60 pounds per square foot, which is 10 pounds *less* than the weight of floor formed of brick arches between the beams, and covered with concrete up to a little above level of tops of beams.

One great advantage of using a buckled plate floor is that the beams are *stayed* laterally, and their tabular capacity can always be used.

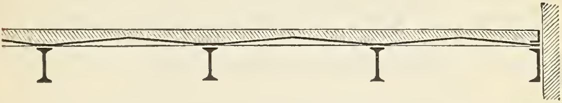
Another advantage is that, by the thorough binding together of the entire floor system, it is likely to be much more rigid than other floors designed for same loads.

In cases where ceilings are necessary, they may be hung to bottom of beams, by means of wire netting, with the usual fastenings; or small joist may be laid transversely between the beams and the ceiling attached thereto.

In ordinary warehouses there is generally no need for ceilings. In such cases, the floor load due to beams, buckled plates, and concrete covering may be taken 50 pounds per square foot, instead of 60 pounds, as given above.



Suppose we have a floor area of 63′ × 18′ inside of walls. If we divide it into four spaces lengthwise by three girders, making the two central spaces 16′.0″, then the two end spaces from centre of girder to centre of wall will be 16′.0″. Into these girders frame floor joist spaced 3′ apart, and running lengthwise, then there will be 6 spaces in the width, of 3′.0″ each. The buckled plates next the wall will be carried on channels of same depth as the floor joist, and around the inner edge of all walls will be a 4″ × 3″ × $\frac{3}{8}$″ angle iron (the 4″ leg set vertically), to confine the concrete. These angles set over the fillets of the buckled plates.



In each panel, then, there will be *five* lines of **I** beams lengthwise of area, and two lines of channels next the long way of the wall. There are also three transverse girders into which are framed the five lines of **I** beams and two lines of channels. Suppose we wish the floor to carry an extraneous load of 100 pounds per square foot, the weight of the buckled plate floor being 60 pounds per square foot, the total load per square foot will be 160 pounds.

Each floor joist will then carry 3′ wide × 160 pounds × 16′ long, or 48 square feet at 160 pounds = 3.84 tons. As these joists are stayed laterally by the buckled plates, we can use the full tabular capacity, and looking in the tables at the 16′ span line, we find that an 8″ **I** beam of iron, 65 pounds per yard, will carry 4.25 tons, and the deflexion is

0.46″. The channel iron against wall will carry but one-half the load on the beams; whence from tables we find that an 8″ channel of iron, 40 pounds per yard, will answer, as its safe load is 2.25 tons, and deflexion 0.50″.

Each transverse girder carries an area of $16' \times 18' =$ 288 square feet. This, at 160 pounds per square foot, has a load of 23.04 tons. The effective span of the girder is about 19′, and looking at 19′ span line in the tables, we find there, if the upper flange is *stayed laterally*, that a 15″ **I** beam of iron, 250 pounds per yard, will do, as it carries 22.73 tons (which is close enough to the load required), and has a deflexion of 0.33″. Or, looking in the Tables of Steel Beams, we find that at 19′ span, a 15″ **I** beam of 20.00 square inches area,—*i.e.*, 202 pounds per yard,—will do, as it carries, when flange is stayed, 25.32 tons.

In framing the 8″ floor joist into the 15″ **I** beam girder, if the top flanges are placed on the same level, the flanges of girder can be considered *stayed*. The joist, however, may be framed into girder 4″ below the bottom of its top flange, in which case the top of concrete is level with top of girder. In this case the flange of girder beam cannot be considered as stayed. Assuming the girder flange $5\frac{1}{2}''$, the ratio of unstayed length to flange width is $\frac{19 \times 12''}{5\frac{1}{2}} =$ about 40; whence tabular loads must be multiplied by 0.88,—that is to say, we can only place an extreme fibre stress of 6.90 tons on the steel beam, instead of 7.8 tons, the tabular fibre stress.

Since f qh $S = M_o$, and

$$M_o = \frac{23.04 \times 20 \times 12}{8} = 691.2 \text{ inch-tons},$$

we get

$$S = \frac{M_o}{f\,qh} = \frac{691.2}{6.9 \times 0.3 \times 15} = 22.26 \text{ square inches};$$

i.e., we need a 15″ steel **I** beam, 22.26 square inches area, or 225 pounds per yard.

The ends of all beams which rest on walls should have

loose bearing plates of iron, say 8″ square, and say $\frac{3}{8}$″ thick. Also there should be riveted on the webs two angle irons, to form "check angles."

An approximate estimate for this floor will read as follows:

1st. Girders with flanges *not stayed.*

	Lbs.
Three steel I beams, 15″ deep, 225 pounds per yard, 19′.6″ long	4,390
Six bearing plates, 12 × $\frac{3}{8}$, 1′.0″ long	90
Twelve "check" angles, 3 × 3 × $\frac{3}{8}$ angles, 21.6 pounds per yard, 12″ long	90
Ten floor joist, 8″ iron I beams, 65 pounds per yard, 16′.6″ long . . . . Ten floor joist, 8″ iron I beams, 65 pounds per yard, 16′.0″ long	7,050
Four floor joist, 8″ iron channels, 40 pounds per yard, 16′.6″ long	880
Four floor joist, 8″ iron channels, 40 pounds per yard, 16′.0″ long	850
Fourteen bearing plates, 8 × $\frac{3}{8}$, 0′.8″ long	95
Twenty check angles on 8″ I beams, 3 × 3 × $\frac{3}{8}$ angles, 0′.6″ long	75
Four check angles on 8″ channels, 3 × 3 × $\frac{3}{8}$ angles, 0′.6″ long	15
One hundred and twenty buckled plates, 36″ square, $\frac{1}{4}$″ thick, at 90 pounds each	10,800
Ninety-six transverse joint covers, 4 × 2 × $\frac{3}{8}$ ⊥'s, 24 pounds per yard, 3′.0″ long	2,305
Two lines, 63′ each, curb angles Two lines, 18′ each, curb angles — 162 linear feet of 4 × 3 × $\frac{3}{8}$ angle iron, 24.9 lbs. per yard .	1350
Connexions of joist to girders contain 72 pieces 3 × 3 × $\frac{3}{8}$ angles, 21.6 pounds per yard, 0′.6″ long .	260
	28,250
Allowance for rivet heads	350
	28,600

	Lbs.		Lbs. per sq. ft.
Girders, bearings, and check angles.	4,570	=	4.03
Floor joist, bearings, check angles, and connexions	9,225	=	8.14
Buckled plates, ⊥ covers, and rivet heads	13,455	=	11.87
Curb angles	1,350	=	1.19
	28,600	=	25.23

Area of floor surface = 63′ × 19′ = 1134 square feet.

2d. Girders with top flanges *stayed*, by the joist being so framed into them that all top flanges are on same level.

On page 216 we found that a 15″ steel **I** beam, 202 pounds per yard, would answer, and previous estimate is changed only in the weight of the three steel girders; whence it would now read

	Lbs.		Lbs. per sq. ft.
Girders, bearings, and check angles.	4,120	=	3.63
Floor joist, bearings, and check angles	9,225	=	8.14
Buckled plates, ⊥ covers, and rivet heads	13,455	=	11.87
Curb angles	1,350	=	1.19
	28,150	=	24.83

Suppose the floor joist are laid in direction of short length of floor area; then there will be 21 spaces of 3′ each,—*i.e.*, 20 floor joist, 19′ span, 2 joist of channels, 19′ span. Each joist will carry 18 × 3 × 160 pounds = 4.32 tons.

Looking at 19′ span line in the tables, we find that we can use a 9″ iron **I** beam, 85 pounds per yard, as this will carry a safe load of 5.05 tons, and has a deflexion of 0.57″. Looking in steel tables, we find that an 8″ **I** beam of steel, 65.75 pounds per yard, will sustain a safe load of 4.91 tons, and has a deflexion of 0.75″. Now, this .75″ deflexion is greater than $\frac{1}{30}''$ per foot of span. If, then, we dare not exceed the limit of $\frac{1}{30}''$ per foot = $0.6\frac{1}{3}''$, we shall have to reduce the safe load to $\frac{0.63}{0.75} \times 4.91 = 0.83 \times 4.91 = 4.08$

tons. This is less than 4.32 tons, the load required to be carried, whence we shall have to use a heavier beam. A 9″ I beam of steel $70\frac{3}{4}$ pounds will answer, since its deflexion being 0.74, we shall have to take $\frac{0.63}{0.74} = 0.85$, its tabular load $= 0.85 \times 5.06 = 4.30$ tons. Thus we can use, having a plaster ceiling, a 9″ iron beam, 85 pounds per yard, or a 9″ steel beam, $70\frac{3}{4}$ pounds per yard.

We will use for the intermediate joist 9″ I beams of steel, $70\frac{3}{4}$ pounds per yard, and for the joist next walls, 9″ iron channels, 42.75 pounds per yard, as they will carry 2.72 tons, a little more than the required load of $\frac{4.32}{2} = 2.16$ tons.

The approximate weight is as follows:

	Lbs.
Twenty 9″ steel I beams, $70\frac{3}{4}$ pounds per yard, 19′.6″ long	9,200
Two 9″ iron channels, $42\frac{3}{4}$ pounds per yard, 19′.6″ long	550
Forty-four bearing plates, $8 \times \frac{3}{8}$, 0′.8″ long	300
Forty-four check angles, $3 \times 3 \times \frac{3}{8}$ angles, 21.6 pounds per yard, 0′.6″	160
Buckled plates, 126, at 90 pounds apiece	11,340
Transverse joint covers, 120′, 4×2 ⊥ $\frac{3}{8}$, 3.0′ long	2,880
Curb angles, as in Estimate 1st	1,350
Allowance for rivet heads	350
	26,130

	Lbs.		Lbs. per sq. ft.
Floor joist, bearings, and check angles	10,210	=	9.00
Buckled plates, ⊥ covers, and rivet heads	14,570	=	12.85
Curb angles	1,350	=	1.19
	26,130	=	23.04

A saving of over 8 per cent. in weight, which is likewise an 8 per cent. saving in dollars and cents, as steel beams cost no more per pound than iron ones.

TRUSSED GIRDERS.

Given a trussed girder whose span centre to centre of end pins is 32 feet, whose depth is $3\frac{1}{3}$ feet centre to centre of chord pins, and carrying a load of 4.0 tons per linear foot. From these dimensions we have tangent $\phi = 10\frac{2}{3} \div 1\frac{1}{3} = 3.20$, and secant $\phi = 11.175 \div 10\frac{2}{3} = 3.35$.

The load on each post, Bb, B′b′, is $42\frac{2}{3}$ tons, since each carries the load due to one-half a panel length on each side of it. This stress of $42\frac{2}{3}$ tons, coming down the post Bb, is resolved at pin b on the chord bars bb′, and on the diagonal bars Ab. On the chord bars bb′ the stress is $42\frac{2}{3} \times \tan.\ \phi = 42\frac{2}{3} \times 3.2 = 136.53$ tons. On the diagonals Ab the stress is $42\frac{2}{3} \times \text{sect.}\ \phi = 42\frac{2}{3} \times 3.35 = 143.06$ tons. This last, coming through the pin A, is resolved on the upper chord, and is $42\frac{2}{3} \times \tan.\ \phi = 136.53$ tons, which is the thrust from A to A′. Whence we have the following stresses:

In upper chord, AB, BB′, B′A′,	$42\frac{2}{3} \times \tan.\ \phi =$	136.53 tons.
In lower chord, bb′,	$42\frac{2}{3} \times \tan.\ \phi =$	136.53 tons.
In diagonal bars, Ab and A′b′,	$42\frac{2}{3} \times \text{sect.}\ \phi =$	143.06 tons.
In vertical posts, Bb and B′b′,	$42\frac{2}{3}$ tons $=$	$42\frac{2}{3}$ tons.

The unit stress f_c for compression is as given before,—viz., $1\frac{2}{3}$ tons $(2 + \phi) = 1\frac{2}{3} \times 3 = 5.00$ tons.

The unit stress for rolled bars is 10 per cent. greater than that given for shape iron in tension,—viz., it is 2.2 tons $(2 + \phi) = 6.60$ tons, ϕ being as formerly given, the ratio of the minimum to the maximum stress in the piece. As this is all dead load in the case under consideration, $\phi = 1$.

For the posts Bb and B′b′. Assume them made of two 9″ channels, laced together. The least radius of gyration is in plane of stress, and is about $3\frac{1}{3}$″; then the length centre to centre of pins being 40″, the ratio $\frac{l}{r} = \frac{40}{3\frac{1}{3}} = 12$. The ends are "pin ends." Whence

reduced unit stress, $p_c = \dfrac{5.00}{1 + \dfrac{1}{20,000}\left(\dfrac{l}{r}\right)^2} = 4.96$ tons;

whence section required is

$$\frac{42\frac{2}{3}}{4.96} = 8.60 \text{ square inches.}$$

Can use two 9″ channels, 46 pounds per yard = 9.20 square inches.

For the lower chord bars, bb′, the stress is 136.53 tons, and the unit stress 6.60 tons; whence section required is 136.53 ÷ 6.60 = 20.68 square inches nett. Use four eye bars, 5″ × 1″ = 20.00 square inches nett.

For the diagonal bars, Ab and A′b′, the stress is 143.06 tons, and the unit stress is 6.60 tons; whence section required is 143.06 ÷ 6.6 = 21.68 square inches nett. Use four eye bars, 5″ × $1\frac{1}{16}$″ = 21.25 square inches nett.

For the upper chord panels AB, BB′, B′A′. In each panel the stress is the same, and is 136.53 tons; but each panel, beside having a longitudinal thrust of 136.53 tons, has also to sustain cross stress, due to a load of 4.0 tons per linear foot. For the end panels AB′, A′B′, we may consider the beam as fixed at the ends B and B′, and merely supported at the ends A and A′. The maximum moment under such a condition is at the end B and B′,—viz., at the pins B and B′,—and is given by

$$M_o = \frac{wl^2}{8} = \frac{4 \times \overline{10\frac{2}{3}}^2}{8} = 56\tfrac{7}{8} \text{ foot-tons} = 682.5 \text{ inch-tons.}$$

The middle panel BB′ we may consider as having "fixed ends," and under such condition the moment at centre of BB′ panel is

$$M_o = \frac{wl^2}{24} = \frac{4 \times \overline{10\frac{2}{3}}^2}{24} = 18.96 \text{ foot-tons} = 227\tfrac{1}{2} \text{ inch-tons.}$$

The unit stress at *a panel point* may be taken as $f_c = 5.00$ tons. The unit stress at the centre of a panel is dependent

upon the ratio of its length to the least radius of gyration. Assuming a 20″ chord, r may be taken $\frac{3}{8} \times 20 = 7.5''$; when

$$\frac{l}{r} = \frac{10\frac{2}{3} \times 12}{7\frac{1}{2}} = \frac{128}{7\frac{1}{2}} = 17$$

Then the reduced unit stress is

$$p_c = \frac{5.00}{1 + c\left(\frac{l}{r}\right)^2}$$

which for "fixed ends" is

$$p_c = \frac{5.00}{1 + \frac{1}{40,000}\left(17\right)^2} = 4\tfrac{2}{3} \text{ tons.}$$

The section required at B and B′ is then

$$S = \frac{1}{5.0}\left\{136.53 + \frac{682.5}{qh}\right\}$$

Now qh may be taken one-third the height $= 6\frac{2}{3}''$; then

$$S = \frac{1}{5.0}\left\{136.53 + \frac{682.5}{6\frac{2}{3}}\right\} = \frac{1}{5.0}\left\{136.53 + 102\ 37\right\}$$

$$= 27.31 + 20.47 = 47.78 \text{ square inches required.}$$

The section required at *centre* of BB′ panel is

$$S = \frac{1}{4\frac{2}{3}}\left\{136.53 + \frac{227\frac{1}{2}}{6\frac{2}{3}}\right\} = \frac{1}{4\frac{2}{3}}\left\{136.53 + 34.12\right\}$$

$$= 29.25 + 7.31 = 36.56 \text{ square inches required.}$$

The section required at B, 47.78 square inches, is then the maximum, and we shall have to make this section constant throughout the chord.

Making a chord 20″ wide, and 21″ deep out to out, the thickness of flange plate must be $\frac{16}{30}$ = say $\frac{1}{2}''$, 16″ being

the width centre to centre of rivets across the flange plate. We can then use

	Sq. in.
One top flange plate, $20 \times \frac{1}{2}$	10.00
Two vertical web plates, $20 \times \frac{1}{2}$	20.00
Four flange angles, $5'' \times 3\frac{1}{2}'' \times$ about $\frac{9}{16}''$, 45 pounds per yard	18.00
Total section used	48.00

For the centre of inertia of this section $S = 48.00$; $wt = 10.00$ square inches; $h + t = 21''$. Then $E = \frac{10}{48} \times \frac{21}{2} = 0.2083 \times 10\frac{1}{2} = 2.18''$.

The estimated weight of this trussed girder is

	Lbs.
Upper chords, battens, lacing, thickening, and bearings	6,750
Two vertical posts, channels, lacing, thickening	460
Four lower chord bars, $5'' \times 1''$	950
Eight diagonal bars, $5'' \times 1\frac{1}{16}''$	2,000
Six pins, $4\frac{1}{2}''$ diameter, and pin nuts . . .	600
	10,760

As the upper chord segment is made in one continuous segment from A to A′, and of constant section, it would be nearer the truth to consider our girder as three spans of a continuous beam, each span being $10\frac{2}{3}$ feet. Taking $wl = 4.0 \times 10\frac{2}{3} = 42\frac{2}{3}$ tons as a factor of shear and moment, we would have (remembering here we call $l = \frac{32}{3} = 10\frac{2}{3}$)

Reactions. A and $A' = \frac{4}{10}\,wl = \frac{4}{10} \times 10\frac{2}{3} = 17.07$.

B and $B' = \frac{11}{10}\,wl = \frac{11}{10} \times 10\frac{2}{3} = 46.93$.

Moments. At A and $A' = 0$.

At B and $B' = \frac{1}{10}\,wl^2 = \frac{1}{10} \times 42\frac{2}{3} \times 10\frac{2}{3} \times 12 = 546.13$ inch-tons.

At centre of AB panel $= \frac{3}{40}\,wl^2 = \frac{3}{40} \times 42\frac{2}{3} \times 10\frac{2}{3} \times 12 = 409.6$ inch-tons.

At centre of BB' panel $= \frac{1}{40}\,wl^2 = \frac{1}{40} \times 42\frac{2}{3} \times 10\frac{2}{3} \times 12 = 136.53$ inch-tons.

Now for stresses due to truss action.

On the posts Bb, B′c′, $= 46.93$ tons.
On the lower chord bars, bb′, $46.93 \times 3.2 = 150.18$ tons.
On the diagonal bars, Ab, A′b′, $46.93 \times 3.35 = 157.27$ tons.

Unit stress in end panel,

$$p_c = \frac{5.00}{1 + \frac{1}{30,000}\left(17\right)^2} = 4.56 \text{ tons.}$$

Unit stress in centre panel,

$$p_c = \frac{5.00}{1 + \frac{1}{40,000}\left(17\right)^2} = 4\tfrac{2}{3} \text{ tons.}$$

Unit stresses at pin points B and B′ $= 5.00$ tons $= f_c$.

For the posts Bb, B′b′, we can take same unit stress as before,—*i.e.*, 4.96 tons; then $46.93 \div 4.96 = 9.46$ square inches required. May use two 9″ channels, 48 pounds per yard $= 9.60$ square inches.

For the lower chord bars we need $150.18 \div 6.6 = 22.75$ square inches nett. Use four bars, $5 \times 1\frac{1}{8} = 22.50$ square inches.

For the diagonal bars we require $157.27 \div 6.6 = 23.83$ square inches nett. Use four bars, $5'' = 1\frac{3}{16}'' = 23.75$ square inches.

For the upper chord we would require, bearing in mind that $qh = \frac{1}{3} \times 20 = 6\frac{2}{3}''$,

At centre of AB panel,

$$S = \frac{1}{4.56}\left\{136.53 + \frac{409.6}{6\frac{2}{3}}\right\} = \frac{1}{4.56}\left\{136.53 + 61.44\right\}$$

$$= 32.93 + 13.47 = 46.40 \text{ square inches.}$$

At panel point B,

$$S = \frac{1}{5.0}\left\{136.53 + \frac{546.13}{6\frac{2}{3}}\right\}$$

$$= 30.04 + 16.36 = 46.40 \text{ square inches.}$$

At centre of panel BB′,

$$S = \frac{1}{4\frac{2}{3}}\left\{136.53 + \frac{136.53}{6\frac{2}{3}}\right\}$$

$$= 32.18 + 4.39 = 36.57 \text{ square inches.}$$

The maximum required is then 46.40 square inches, which we shall make constant from A to A′. Use

	Sq. in.
One $20 \times \frac{1}{2}$ flange plate	10.00
Two $20 \times \frac{1}{2}$ web plates	20.00
Four $5 \times 3\frac{1}{2} \times \frac{1}{2}$ angles, 41.1 pounds per yard	16.44
Total section used	46.44

The estimated weight would then be

	Lbs.
Upper chord, battens, lacing, thickening, and bearings	6,500
Vertical posts, channels, lacing, and thickening	500
Four lower chord bars, $5 \times 1\frac{1}{8}$	1,050
Eight diagonal bars, $5 \times 1\frac{3}{16}$	2,250
Six pins, $4\frac{1}{2}''$ diameter, and pin nuts . . .	600
	10,900

The estimated weight of a single-webbed plate girder to carry the same load was 9390 pounds (see page 204), and that of a box plate girder was 11,080 (see page 211); whence we have

Estimated weight of plate girder = 9,390 lbs.
Estimated weight of box girder = 11,080 lbs.
Estimated weight of trussed girder = 10,900 lbs.

from which it is seen that the single-webbed plate girder is the most economical in weight; it is likewise the most economical of construction.

Trussed girders of iron, to carry lighter loads, may have their upper chords made of a pair of channels connected by

a top flange plate, and the diagonals may often be made of square bars, with sleeve nuts.

The central panel should generally, no matter what the loading, have adjustable diagonal bars, for though not needed, theoretically, for an uniformly distributed load, yet are of service to transmit the unequal loads caused by the wall being "run up" irregularly.

Small span trussed girders may also be built of timber, as per sketch. Each half of the beam takes cross stress due to load on the distance $\frac{l}{2}$, as well as the direct thrust caused by the truss rod.

If T denote the direct thrust in tons on the beam; b, the breadth of the beam; h, the height; l, the length of beam centres of supports, then the panel length is $\frac{l}{2}$; and w the load in tons per linear foot.

Then area of beam required is given by

$$bh = S = \frac{1}{f}\left\{\frac{M}{qh} + T\right\}$$

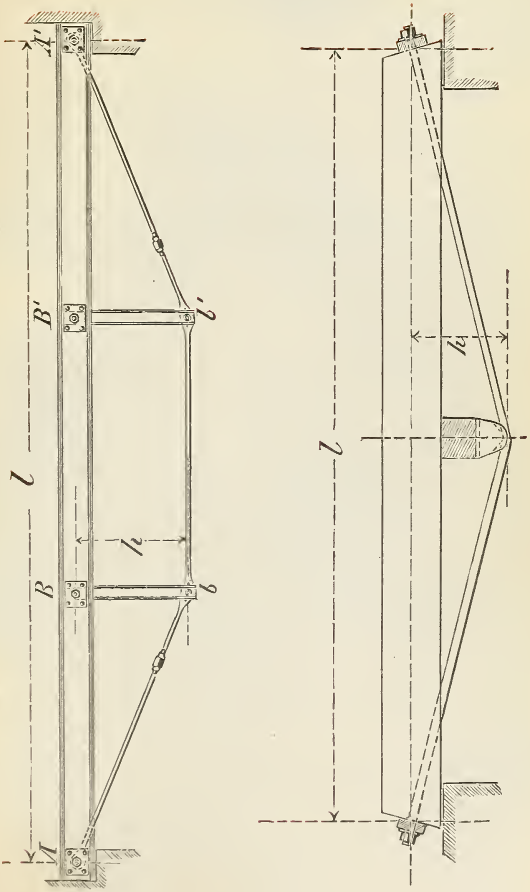
Taking l the total length in feet, as per sketch, and w the load in tons per foot, the maximum moment in *inch-tons*, $M = \frac{3}{8} \cdot wl^2$; also $qh = \frac{h}{6}$, since for the rectangular section $q = \frac{1}{6}$, and f may be taken 1200 pounds per square inch $= 0.6$ tons.

Then area of beam required is

$$S = bh = \frac{1}{0.6}\left\{\frac{18}{8}\,\frac{wl^2}{h} + T\right\} = \frac{1}{0.6}\left\{\frac{9}{4}\cdot\frac{wl^2}{h} + T\right\}$$

$$= 3.75\,\frac{wl^2}{h} + \frac{T}{0.6}$$

where w is the load per linear foot in tons; l, the total span in feet; h, the height in inches; b, the breadth in inches; and T, the thrust in tons.


A'
B'
b'
l
h
B
b
A
h
l

FLITCH BEAMS.

A flitch beam is one in which are combined wooden beams and rectangular iron plates, the iron plates being so bolted together through the timbers as to prevent lateral deflexion of the former.

Let there be n beams of timber, of h depth and b thickness each, and m plates of iron, of h depth and t thickness each, and so bolted to the wooden beams that the full fibre stress f_c may be used on the iron plates.

The moment of inertia of a rectangular section is $I = \frac{b h^3}{12} = \frac{S h^2}{12}$; and for symmetrical sections q being equal to $\frac{2 I}{h^2 S}$, then q for rectangular sections $= \frac{1}{6}$.

Now, from equation $M_o = f\,qh\,S$, we may determine the capacity of a flitch beam.

It must, however, be borne in mind that the *deflexions* of each material under its proportion of the total load on the combined beam should be *the same*, and as the deflexions of two beams of different material are inversely as their moduli of elasticity, the fibre stresses used should be in same proportion. The ratio of the modulus of elasticity of iron is to that of wood as 18 to 1; whence, if we take f_c for iron as 6.0 tons per square inch, that for wood should be $\frac{1}{18}$ of 6.0 tons,—viz., $\frac{1}{3}$ of a ton per square inch. Whence, for the timber beams,

$$M_o = f\,qh\,S = \tfrac{1}{3} \times \tfrac{1}{6} \times h\,S = \frac{h\,S}{18}$$

but $$S = nbh$$

then $$M_o = \frac{nbh^2}{18} \qquad (1)$$

And for the iron plates,

$$M_o = f\,qh\,S = 6.0 \times \tfrac{1}{6} \times h\,S = hS$$

but

$$S = mth$$

whence

$$M_o = mth^2 \qquad (2)$$

Now, adding these, we get

$$\Sigma\,M_o = \frac{nbh^2}{18} + mth^2 = h^2\left\{\frac{nb}{18} + mt\right\} \qquad (3)$$

If w be the load per linear unit on the beam, and l the effective span, then the maximum moment is at centre, and is $\frac{wl^2}{8}$.

Equating this to (3), we get

$$\frac{wl^2}{8} = h^2\left\{\frac{nb}{18} + mt\right\}$$

or

$$wl = \frac{8\,h^2}{l}\left\{\frac{nb}{18} + mt\right\} \qquad (4)$$

denoting by w, the load in tons per linear foot, and l′ the length of beam in feet; then wl will be the total load in tons, and may be written W, and $l = 12\,l'$, and equation (4) will become

$$W = \frac{8\,h^2}{12\,l'}\left\{\frac{nb}{18} + mt\right\} = \frac{2}{3}\cdot\frac{h^2}{l'}\left\{\frac{nb}{18} + mt\right\} \qquad (5)$$

In (5) it is seen that if $mt = \frac{nb}{18}$,—*i.e.*, if the total thickness of the iron plates be $\frac{1}{18}$ of the total breadth of the timber beams,—that the total load carried becomes

$$W = \frac{2}{27}\cdot\frac{h^2}{l'} \qquad (6)$$

i.e., total load in tons $= \frac{2}{27}$ of the ratio of the square of the

depth in inches to the span in feet. It is also evident that if we want to *double* the strength of a wooden girder, we should add flitch plates whose aggregate thickness $= \frac{1}{18}$ the breadth of the girder.

If r denote the ratio $\frac{mt}{nb}$; then $mt = r \cdot nb$, and, substituting this in (5), we get

$$W = \frac{2}{3} \cdot \frac{h^2}{l'} \left\{ \frac{nb}{18} + rnb \right\} = \frac{2}{3} \cdot \frac{h^2}{l'} nb \left\{ \frac{1}{18} + r \right\}$$

$$= \frac{2}{3} \cdot \frac{h^2}{l'} \frac{nb}{18} \left\{ 1 + 18\,r \right\} \qquad (7)$$

Now, suppose we have two pieces of $7'' \times 14''$ forming a girder of 20 feet effective span, and it is required to add iron plates in order to increase its strength $\frac{2}{3}$; then $18\,r$ should equal $\frac{2}{3}$,—*i.e.*, $r = \frac{1}{27}$; whence $mt = \frac{nb}{27} = \frac{14}{27} = 0.519''$. Whence

$$W = \frac{2 \times \overline{14}^2 \times 14}{3 \times 20 \times 18} \left\{ 1 + \frac{2}{3} \right\} = 5.08 \times 1\tfrac{2}{3} = 8.47 \text{ tons};$$

or, taking separately, the wooden beams will stand 5.08 tons, and the iron plates $\frac{2}{3} \times 5.08 = 3.39$ tons.

Thus, an iron plate about $\frac{1}{2}'' \times 14''$ placed between the two wooden beams of $7'' \times 14''$ will add $\frac{2}{3}$ to the strength of the wooded beams, and their deflexions will be alike. The fibre stresses under the above loads will be, on the iron 6.0 tons per square inch, and on the timber $\frac{1}{3}$ of a ton per square inch.

As regards the deflexion. The expression which gives the centre deflexion for the wooden beams is

$$\Delta = \frac{5}{384} \left\{ \frac{wl^3}{E\ I} \right\} \qquad (8)$$

where W = total load borne by the timber beams.
l = span of beams in inches.
E = modulus of elasticity of timber = $722\frac{2}{9}$ tons.
I = moment of inertia of the wooden beams = $\frac{1}{12} \times nb \times h^3$, where nb = aggregate breadth of beams, and h = height in inches.

Also, the deflexion for the iron flitches is given by

$$\Delta_1 = \frac{5}{384}\left\{\frac{W'l^3}{E'\,I'}\right\} \qquad (9)$$

where W' = total load borne by the flitches.
l = span of beam in inches.
E' = modulus of elasticity of iron = 13,000 tons.
I' = moment of inertia of the iron plates = $\frac{1}{12} \times mt \times h^3$, where mt = width of flitches, and h = height in inches.

Now, by the hypothesis, the deflexion of each material of the compound beam is the same,—*i.e.*, $\Delta = \Delta_1$; therefore the ratio

$$\frac{\Delta}{\Delta_1} = \frac{W}{W'}\left\{\frac{E'\,I'}{E\,I}\right\} = 1 \qquad (10)$$

whence

$$W = W'\left\{\frac{nb}{18\,mt}\right\} \qquad (11)$$

and

$$W' = W\left\{\frac{18\,mt}{nb}\right\} \qquad (12)$$

The value of (8) for our example, in which we have $l = 20' = 240''$; $h = 14''$; $nb = 14''$; $E = 722\frac{2}{9}$ tons;

$$I = \frac{nbh^3}{12} = \frac{14 \times \overline{14}^3}{12} = 3201\tfrac{2}{3}, \text{ is}$$

$$\Delta = \frac{5}{384}\left\{\frac{\overline{240}^3}{722\frac{2}{9} \times 3201\frac{2}{3}}\right\} . W = 0.0788\ W$$

but

$$W = 5.08$$

whence

$$\Delta = 0.0788 \times 5.08 = 0.40''$$

And the value of (9) should be the same. Here we have $l = 20' = 240''$; $h = 14''$; $mt = .519'' = \frac{14}{27}$; $E = 13{,}000$ tons; and $I = \frac{.519 \times 14^3}{12} = \frac{14 \times \overline{14}^3}{12 \times 27} = 118.58$

Then

$$\Delta_1 = \frac{5}{384} \left\{ \frac{\overline{240}^3}{13{,}000 \times 118.58} \right\} W' = 0.1182\ W';$$

but

$$W' = 3.39 \text{ tons,}$$

whence

$$\Delta_1 = 0.1182 \times 3.39 = 0.40''$$

From (12) we see that

$$mt = \frac{W'}{W} \left\{ \frac{nb}{18} \right\} \qquad (13)$$

Whence the following *rule:* Having given in a certain span, wooden beams of nb aggregate thickness, whose safe load at $\frac{1}{3}$ ton fibre stress is W, if we wish to add W′ tons to the capacity of these beams, by adding iron flitches of same depth, the thickness of such flitches is given by

$$mt = \frac{W'}{W} \left\{ \frac{nb}{18} \right\}$$

Example. Given $nb = 14''$, $W = 5.08$ tons; and we wish to add $W' = 3.39$ tons; then

$$mt = \frac{3.39}{5.08} \left\{ \frac{14}{18} \right\} = \frac{2}{3} \times \frac{7}{9} = \frac{14}{27} = 0.519''$$

Also, having given in a certain span, wooden beams of nb aggregate breadth, whose safe load is W, if we add flitch plates of mt thickness, we will add to the capacity of the beams

$$W' = 18\ W \left\{ \frac{mt}{nb} \right\}$$

EXAMPLE. Given mt $= 0.519$, nb $= 14''$; whence

$$\frac{mt}{nb} = \frac{1}{27}; \text{ also } W = 5.08 \text{ tons.}$$

Then

$$W' = \frac{18}{27} \times 5.08 = \frac{2}{3} \times 5.08 = 3.39 \text{ tons.}$$

If we wish to *double* the strength of the wooden beam by addition of flitch plates, the thickness of such plates will be given by mt $= \frac{nb}{18}$, since $W = W'$.

EXAMPLE. Given nb $= 14$, $W = 5.08$ tons; then

$$mt = \frac{14}{18} = \frac{7}{9} = 0.778''$$

which we can check by (7); for, in order to double the strength of the beam, we should have $18\ r = 1$, then $r = \frac{1}{18}$; and, since $r = \frac{mt}{nb}$, then mt $= r \cdot nb = \frac{14}{18} = 0.778''$.

Substituting $18\ r = 1$ in (7), and we get

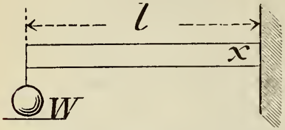
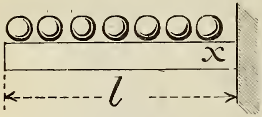
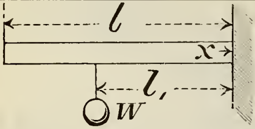
$$W = \frac{2}{3} \times \frac{h^2}{l'} \times \frac{nb}{18} \times 2$$

$$= \frac{2 \times \overline{14}^2 \times 14 \times 2}{3 \times 20' \times 18} = 5.08 \times 2 = 10.16 \text{ tons.}$$

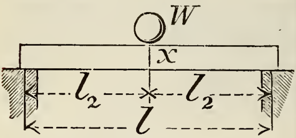
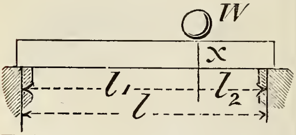
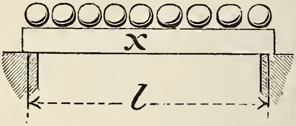
BENDING MOMENTS AND SHEARING FORCES

For different loads and supports.

BEAMS FIXED AT ONE END.

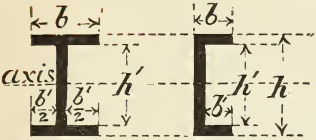
Diagram.	Maximum bending moment at X.	Maximum shearing force.	Loading.
	Wl	W	Load at end.
	$\frac{Wl^2}{2}$	Wl	Uniformly loaded with W lbs. per lineal foot.
	Wl_1	W	Eccentric Loading.

BEAMS WITH SUPPORTED ENDS.

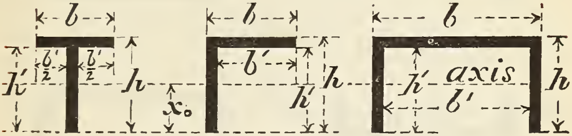
Diagram.	Maximum bending moment at X.	Maximum shearing force.	Loading.
	$\frac{Wl}{4}$	$\frac{W}{2}$	Load at centre.
	$\frac{Wl_1l_2}{l}$	$\frac{Wl_1}{l}$ and $\frac{Wl_2}{l}$	Eccentric Loading.
	$\frac{Wl^2}{8}$	$\frac{Wl}{2}$	Uniformly distributed load of W lbs. per lineal foot.

MOMENTS OF INERTIA

For Simple Shapes.

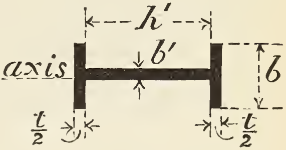


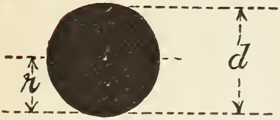
$$I = \frac{bh^3 - b^1 h^{1^3}}{12}$$



$$I = \frac{(bh^2 - b^1 h^{1^2})^2 - 4 \, . \, bh \, . \, b^1 h^1 (h - h^1)^2}{12 \, (bh - b^1 h^1)}$$

$$J = \tfrac{1}{12} (tb^3 + h^1 b^{1^3})$$



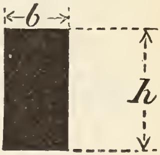


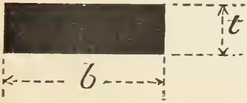
$$I = .7854 \; r^4 = \frac{Ad^2}{16}$$

Where A = area of circle.

$$I = \frac{bh^3}{12} = \frac{Ah^2}{12}$$

Where A = area = bh.





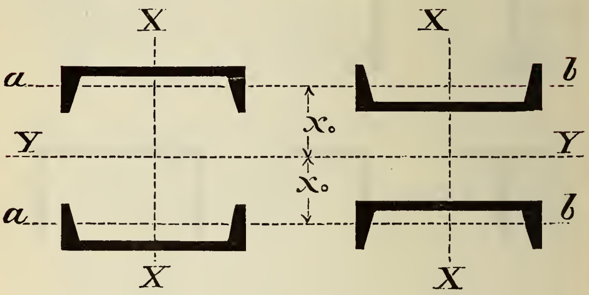
$$I = \frac{bt^3}{12} = \frac{At^2}{12}$$

Where A = area = bt.

MOMENTS OF INERTIA

For Compound Shapes.

Two channels, with lacing, arranged thus:



Line ab = neutral axes of channels.
S = area of each channel.
x_o = distance from neutral axis of channel to axis of compound shape YY.
J = least moment of inertia of the channel.
I = greatest moment of inertia of the channel.

Moment of inertia, axis YY,

$$= 2\left[J + x_o^2 S\right]$$

Radius of gyration, axis YY,

$$= \sqrt{\frac{2\,[J + x_o^2\,S]}{2\,S}} = \sqrt{x_o^2 + \frac{J}{S}} = \sqrt{x_o^2 + r_J^{\,2}}$$

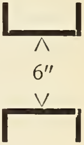
Moment of inertia, axis XX,

$$= 2\,I$$

Radius of gyration, axis YY,

$$= \sqrt{\frac{2\,I}{2\,S}} = \sqrt{\frac{I}{S}}. = r_I$$

Required the least radius of gyration of a column formed of two 10″ channels, 60 pounds per yard, placed 6″ apart, back to back of webs, as shown in figure.



The distance from back of a 10″ channel, 60 pounds to the neutral axis of such channel, is given by the Table of Properties of Channels as 0.69″; therefore the distance from neutral axis of channel to neutral axis of compound shape is $\frac{6''}{2} + 0.69'' = 3.69''$. We also find the radius of gyration of the channel r_J to be 0.79 (see column 13 of Table of Properties above referred to).

Our formula is

$$r = \sqrt{x_o^2 + \frac{J}{S}} = \sqrt{x_o^2 + r_J^2}$$

which for the 10″ channel post is

$$r = \sqrt{3.69^2 + 0.79^2} = 3.77$$

The radius of gyration when the axis is perpendicular to web is, for the 10″ channel, 60 pounds per yard, as per table, 3.69″.

Thus, we find that the column is slightly weaker in the direction of plane of channels than in a direction perpendicular to such plane.

Suppose we wish to form a post of two 12″ channels, 90 pounds per yard, and that we desire to know how far apart in the clear to place these channels in order that both radii of gyration be the same. We simply equate the expressions

$$\sqrt{x_o^2 + r_J^2} \text{ and } r_I;$$

whence

$$x_o^2 = r_I^2 - r_J^2 = (r_I + r_J)(r_I - r_J)$$

Now for the 12″ channel, 90 pounds, the table gives us $r_I = 4.49$; $r_J = 0.89$.

Therefore $\quad 4.49 + 0.89 = 5.38$

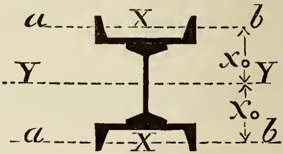
and $\quad 4.49 - 0.89 = 3.60$

and $\quad x_o^2 = 5.38 \times 3.60 = 19.37$

therefore $\quad x_o = \sqrt{19.37} = 4.40''$

Now the distance from back of 12″ channel, 90 pounds to its neutral axis, is, as per table, 0.84. Therefore distance of back of channel from centre of compound shape $= x_o - 0.84 = 4.40 - 0.84 = 3.56''$. Thus channels should be placed apart $2 \times 3.56 = 7.12''$, say 7 inches in the clear.

TWO CHANNELS AND I BEAM.



ab = neutral axis of channel.
S_1 = area of channel.
S_2 = area of beam.
J_1 = least moment of inertia of channel.
J_2 = least moment of inertia of beam.
I_1 = greatest moment of inertia of channel.
I_2 = greatest moment of inertia of beam.

Moment of inertia, axis YY,

$$= I_2 + 2\,[J + x_o^2 \cdot S_1]$$

Radius of gyration, axis YY,

$$= \sqrt{\left[\frac{I_2 + 2\,[J + x_o^2 \cdot S_1]}{2\,S_1 + S_2}\right]}$$

Moment of inertia, axis XX,

$$= J_2 + 2\,I_1$$

Radius of gyration, axis XX,

$$= \sqrt{\frac{J_2 + 2\,I_1}{2\,S_1 + S_2}}$$

Required the moments of inertia of a column, formed as above, of two 10″ channels, 48 pounds per yard, and one 12″ **I** beam, 125 pounds per yard.

First, axis being YY.

Maximum moment of inertia of 12″ **I**, 125 pounds = 279.0. Least moment of inertia of 10″ channel, 48 pounds = 2.40; distance from back of channel to neutral axis = 0.59; whence x_0 = one-half depth of beam + 0.59 = 6.59.

Therefore total moment of inertia of column, the axis being YY, is

$$279.0 + 2\left[2.40 + (6.59)^2 \times 4.8\right]$$

$$= 279.0 + 2 \times 208.45 = 695.90$$

The area of compound section = 12.5 □″ + 2 × 4.8 = 22.1 □″. Therefore radius of gyration, axis being as above, is

$$= \sqrt{\frac{699.74}{22.1}} = 5.611''$$

Second, the axis being XX.

Least moment of inertia of 12″ **I** beam, 125 pounds =	14.50
Twice maximum moment of inertia of 10″ channel, 48 pounds =	130.00
Moment of inertia of compound section, axis XX =	144.50

The radius of gyration is

$$\sqrt{\frac{144.50}{22.1}} = 2.56''$$

Thus, around the axis YY the compound section, formed of one 12″ beam, 125 pounds, and two 10″ channels, 48 pounds, is more than twice as strong as around the axis XX, provided, of course, the condition of ends of columns is the same; as, for example, both fixed ends.

BEARING OF

GIRDERS ON BRICK WALLS.

The pressure on a brick wall should not exceed 8 tons per square foot; hence when beams are used for floor joist, their bearings on wall should be so proportioned as not to exceed the above limit. This is conveniently done by means of a loose $\frac{3}{8}''$ plate of wrought iron.

The ends of girders and floor joist should have "check angles" at their wall ends, thus checking the walls from falling outwards in case of fire.

The depth which the beam extends in the wall must not be less than 8 inches.

The thrust of the brick arches is taken up by tie rods $\frac{3}{4}$ to 1 inch in diameter, spaced from 5 to 8 apart, the holes for which are punched in middle of web.

GIRDERS FORMED OF BEAMS

Placed side by side, and beams placed one over the other, and riveted along the flanges.

In supporting heavy walls, the beams can be placed side by side, or be coupled, as in the following sketches.

The width of wall to be supported sometimes prevents the use of more than two beams under them; and in such cases, if two beams cannot be found sufficient to carry the load, two coupled beams can be used, as shown by Fig. 2; or, if they be found insufficient, two sets of three beams each, placed one over the other, can be used. (See Fig. 3.) The coupled and trebled beams are used in lieu of plate girders. If plate girders be used, they would be with a single web, and the wide top flange necessary to carry wall would make the use of heavy vertical stiffeners a necessity.

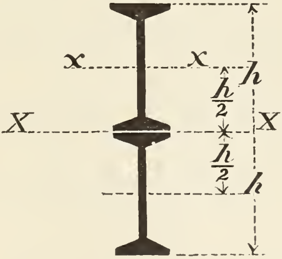
In using coupled and trebled beams, cast-iron separators

No. 1

No. 2

No. 3

are needed, and are generally made of depth of the compound shape. Between brick work and top of beams should be placed a slate or granite plate $2\frac{1}{2}''$ to $5''$ thick, to get an even bearing for wall. This plan of carrying heavy walls is much used by the United States Government in the Public Buildings.



Two **I** beams coupled, as in the above sketch. Required the moment of inertia? Both beams being of same depth and weight.

Let h = height of beam, then $\frac{h}{2}$ = distance from centre of inertia of single beam to centre of inertia of compound

shape. Let S = area of one beam, then $2\ S$ = area of compound section.

I = moment of inertia of each single beam, axis XX.

I_c = moment of inertia of compound shape, axis XX.

Then

$$I_c = 2\left[I + \frac{h^2}{4}S\right] = 2\,I + \frac{h^2\,S}{2}$$

but

$$\frac{h^2\,S}{2} = \frac{I}{q}$$

$$\therefore \quad I_c = 2\,I + \frac{I}{q} = \left(\frac{2\,q + 1}{q}\right) I$$

Now, for the standard or minimum rolls of each **I** beam, q has the average value, 0.33; whence

$$\frac{2\,q+1}{q} = \frac{2 \times 0.33 + 1}{0.33} = 5$$

$$\therefore \quad I_c = 5\,I$$

If R_c be the modulus of this compound shape, then

$$R_c = \frac{2\,.\,I_c}{2\,.\,h} = \frac{I_c}{h} = \frac{5\,I}{h} = 2.5\ R$$

where R is the modulus for the single beam. Whence the moment of resistance of the coupled beams is $2\frac{1}{2}$ times that for a single beam.

For maximum rolls of a beam, q has the average value of 0.3; whence

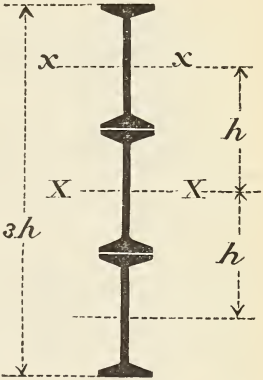
$$\frac{2\,q+1}{q} = 5.33, \text{ and } I_c = 5.33\ I$$

The modulus R_c then becomes 2.67 . R. Thus, for the heavier rolls of beams, the moment of resistance of the coupled beams is 2.67 times that for a single beam.

Comparing the coupled beams with two beams of same depth and weight, placed side by side, the coupled beams

are 1.25 stronger than if the two beams be placed side by side, if the sections be the minimum rolls; and 1.33 times stronger if the sections be the heavier rolls.

The rivets connecting the flanges together should be $\frac{7}{8}''$ or $\frac{3}{4}''$ diameter, dependent upon the thickness of the flanges, and the pitch should be about 6″ or 8″ staggered. At ends of beams the pitch of rivets should be from 3″ to 4″ for a length of twice the depth of the compound shape.



Three beams riveted together as in adjoining sketch. Each beam being of same depth and weight.

Let h = height of each beam; then h is the distance from centre of inertia of outside beams to centre of inertia of compound shapes.

Let S = area of each beam; then 3 S = area of compound section.

I = moment of inertia of each beam, when referred to its own neutral axis.

I_c = moment of inertia of compound shape.

Then

$$I_c = I + 2\left[I + h^2 S\right] = 3\,I + 2\,h^2 S$$

but

$$2\,h^2 S = \frac{4\,I}{q}$$

$$\therefore \quad I_c = 3\,I + \frac{4\,I}{q} = \left(\frac{3q + 4}{q}\right) I$$

For minimum rolls, $I_c = 15$ I.
For maximum rolls, $I_c = 16$ I.
For minimum rolls, $R_c = 5$ R.
For maximum rolls, $R_c = 5.33$ R.

Comparing the trebled beams with 3 beams of the same depth and weight, placed side by side, the trebled beams are 1.66 times stronger than if the 3 beams be placed side by side, if the beams be the minimum rolls; and 1.78 times stronger if the sections be the maximum rolls.

FIRE-PROOF FLOORS.

The dead weight of a fire-proof floor, comprising 4″ brick arches, levelled up to top of beam with concrete, the ceiling and the flooring will run about 70 pounds per square foot of floor surface.

The live weight, equal to a dense crowd of people, is taken at 80 pounds per square foot.

The total weight is then assumed 150 pounds per square foot, exclusive of weight of beams themselves.

The following loads are *exclusive* of weight of arches and beams:

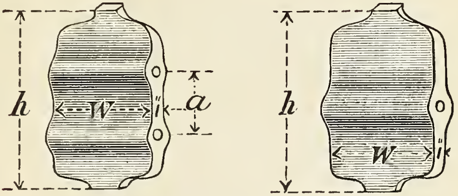
	Lbs. per square foot.
Dense crowd of people	80
Floors of houses	50
Theatres, churches	80
Ball rooms	90
Warehouses	250
Factories	200 to 450
Snow, 30 inches deep	15

	Lbs. per cubic foot.
Brick walls	112
Stone walls	116 to 144

STANDARD SEPARATORS

OF

POTTSVILLE IRON AND STEEL CO.



Width, in inches.	Height, in inches.	Number of bolts.	Length of bolt, in inches.	Distance apart, in inches.	Weight of beam per yard, in pounds.	Weight of separators and bolts, in pounds.
5	15	2	7¼	8	200	22.29
4½	15	2	6½	8	150	20.06
4⅝	12	2	7⅛	6	170	17.2
4¼	12	2	6¼	6	125	16.06
4½	10½	1	6½	In centre	135	13.45
4	10½	1	6	"	105	11.97
3⅝	10½	1	5½	"	90	10.82
4	9	1	6⅛	"	90	10.88
3⅞	9	1	5¾	"	85	8.5
3⅞	9	1	5⅝	"	70	8.4
3⅞	8	1	5⅞	"	80	7.88
3⅝	8	1	5¼	"	65	7.5
3¼	7	1	5⅛	"	65	6.8
3¼	7	1	4⅞	"	55	6.76
3	6	1	4⅝	"	50	5.73
2⅞	6	1	4⅜	"	40	5.2

All standard separators are 1″ thick.

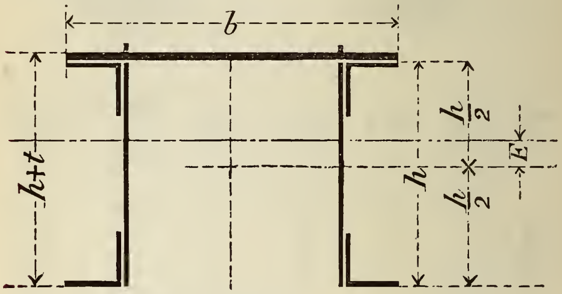
All separator holes are ⅞″ diameter for ¾″ bolts.

All standard separators made for close girders, except when ordered otherwise.

POSITION OF CENTRE OF INERTIA OF A COMPOUND SECTION.

When a compound section is formed of vertical plates, to which are attached angle irons at their extremities, if the angles are similar and similarly placed, the centre of inertia is at the centre of the vertical plates. If a flange plate be added to one side of the section, the position of the centre of inertia will be shifted from the centre, upwards if the plate be on top, downwards if the plate be on the bottom.

For the amount of such moving of the centre of inertia from centre of vertical plates



Let S = total area of section.
h = vertical height out to out of angle iron flanges.
b = breadth of top flange plate.
t = thickness of top flange plate.
E = distance of centre of inertia of compound section from the centre of vertical plates; in other words, the eccentricity of the centre of inertia.

Then

$$E = \frac{bt}{S}\left[\frac{h+t}{2}\right]$$

i.e., the eccentricity E = the ratio of area of top plate to total area of section multiplied by one-half the total height of the section.

In well-designed chords of above "make up," the value of r is about $\frac{3}{8}$ the height, and the value of q about 90 per

cent. of r,—viz., about $\frac{1}{3}$. (For *very* heavy sections q is about 0.30.) For purposes of calculation, r may be taken $\frac{3}{8}$ h, and $q = \frac{1}{3}$; whence $qh = \frac{h}{3}$.

In some very favorable sections q may run as high as 0.38, and r from 0.40 to 0.42 times the height.

COLUMNS AND POSTS.

The table of the ultimate and safe strength of hollow, cylindrical wrought- and cast-iron columns is given on page 248. It is computed by Gordon's formula for varying values of the ratio of length to diameter. The factor of safety for cast-iron columns has been taken at 6, and that for wrought-iron columns at 4. It is assumed that the ends are fixed in direction, such as having planed bearings on capitals and bases.

The table on the ultimate and safe strength of wrought-iron columns is computed according to Rankine's formula for varying values of the ratio of the length to the least radius of gyration, and for the three conditions of square end bearings, one square end bearing and the other pin end, and for both ends with pin bearings. The factor of safety used in the tables for safe strength is 5. If the column be subjected to loads without vibration, the factor could be 4.

To illustrate the use of this table, suppose we wish the ultimate strength of 15″ I beam, 125 pounds per yard, when used as a post, its ends being fixed, and having an unsupported length of 8′ 6″.

Referring to the Tables of the Properties of I Beams, we find that the least radius of gyration, r_J, is given as 1.03″; the length being 8′ 6″ = 102″; the ratio $\frac{l}{r} = \frac{102}{1.03} =$ say 100; for which, on looking at the table, we find the ultimate strength to be 32,000 pounds per square inch. The section of the 15″ beam being 12.5 □″, the ultimate strength is then $12\frac{1}{2} \times 32{,}000$ pounds = 400,000 pounds.

Strength of Hollow, Cylindrical

WROUGHT- AND CAST-IRON COLUMNS

When fixed at the ends.

Computed by Gordon's formula, $P = \dfrac{fS}{1 + c\left(\dfrac{l}{h}\right)^2}$

Let P = ultimate strength, in pounds, per square inch.
S = sectional area, in square inches.
$\left.\begin{array}{l} l = \text{length of column,} \\ h = \text{diameter of column,} \end{array}\right\}$ both in same units.
$\dfrac{l}{h}$ = ratio of length to diameter.
$f = \begin{cases} 40{,}000 \text{ pounds for wrought iron.} \\ 80{,}000 \text{ pounds for cast iron.} \end{cases}$
$C = \frac{1}{3000}$ for wrought iron, and $\frac{1}{800}$ for cast iron.

For cast iron, $$P = \frac{80{,}000\ S}{1 + \frac{1}{800}\left(\frac{l}{h}\right)^2}$$

For wrought iron, $$P = \frac{40{,}000\ S}{1 + \frac{1}{3000}\left(\frac{l}{h}\right)^2}$$

Ratio of length to diameter, $\frac{l}{h}$	Maximum load, per square inch.		Safe load, per square inch.	
	Cast iron.	Wrought iron.	Cast iron, factor of 6.	Wrought iron, factor of 4.
8	74,075	39,164	12,346	9791
10	71,110	38,710	11,851	9677
12	67,796	38,168	11,299	9542
14	64,256	37,546	10,709	9386
16	60,606	36,854	10,101	9213
18	56,938	36,100	9,489	9025
20	53,332	35,294	8,889	8823
22	49,845	34,442	8,307	8610
24	46,510	33,556	7,751	8389
26	43,360	32,642	7,226	8161
28	40,404	31,712	6,734	7928
30	37,646	30,768	6,274	7692
32	35,088	29,820	5,848	7455
34	32,718	28,874	5,453	7218
36	30,584	27,932	5,097	6983
38	28,520	27,002	4,753	6750
40	26,666	26,086	4,444	6522
42	24,962	25,188	4,160	6297
44	23,396	24,310	3,899	6077
46	21,946	23,454	3,658	5863
48	20,618	22,620	3,436	5655
50	19,392	21,818	3,262	5454
52	18,282	21,036	3,047	5259
54	17,222	20,284	2,870	5071
56	16,260	19,556	2,710	4889
58	15,368	18,856	2,561	4714
60	14,544	18,180	2,424	4545

Ultimate and Safe Strength of

WROUGHT-IRON COLUMNS.

p = ultimate strength per square inch.
l = length of column, in inches.
r = least radius of gyration, in inches.

For square end bearings,

$$p = \frac{40,000}{1 + \frac{1}{40,000}\left(\frac{l}{r}\right)^2}$$

For one pin and one square bearing,

$$p = \frac{40,000}{1 + \frac{1}{30,000}\left(\frac{l}{r}\right)^2}$$

For two pin bearings,

$$p = \frac{40,000}{1 + \frac{1}{20,000}\left(\frac{l}{r}\right)^2}$$

For safe working load on these columns, use a factor of 4 when used in buildings, or when subjected to dead load only; but when used in bridges the factor should be 5.

$\frac{l}{r}$	Ultimate strength, in pounds, per square inch.			$\frac{l}{r}$	Safe strength, in pounds, per square inch, factor of 5.		
	Square ends.	Pin and square end.	Pin ends.		Square ends.	Pin and square ends.	Pin ends.
10.0	39,944	39,866	39,800	10.0	7989	7973	7960
15.0	39,776	39,702	39,554	15.0	7955	7940	7911
20.0	39,604	39,472	39,214	20.0	7921	7894	7843
25.0	39,384	39,182	38,788	25.0	7877	7836	7758
30.0	39,118	38,834	38,278	30.0	7821	7767	7656
35.0	38,810	38,430	37,690	35.0	7762	7686	7538
40.0	38,460	37,974	37,036	40.0	7692	7595	7407
45.0	38,072	37,470	36,322	45.0	7614	7494	7264
50.0	37,646	36,928	35,525	50.0	7529	7386	7105
55.0	37,186	36,336	34,744	55.0	7437	7267	6949
60.0	36,697	35,714	33,898	60.0	7339	7143	6780
65.0	36,182	34,478	33,024	65.0	7236	6896	6605
70.0	35,634	34,384	32,128	70.0	7127	6877	6426
75.0	35,076	33,682	31,218	75.0	7015	6736	6244
80.0	34,482	32,966	30,288	80.0	6896	6593	6058
85.0	33,883	32,236	29,384	85.0	6777	6447	5877
90.0	33,264	31,496	28,470	90.0	6653	6299	5694
95.0	32,636	30,750	27,562	95.0	6527	6150	5512
100.0	32,000	30,000	26,666	100.0	6400	6000	5333
105.0	31,357	29,250	25,786	105.0	6271	5850	5157

AVERAGE ULTIMATE CRUSHING LOADS.

TIMBER.	Weight per cubic foot.	Lbs. per sq. in.
Ash	48	8600
Beech, unseasoned	53	7700
Beech, seasoned	43	9300
Cedar, unseasoned	56	5700
Cedar, seasoned	50	6500
Oak, unseasoned	54	4200
Oak, seasoned	67	6000
Pine, pitch		6800
Pine, yellow, unseasoned		5300
Pine, yellow, seasoned		5400
Pine, white, unseasoned	35	5000
Poplar, unseasoned		3100
Poplar, seasoned		5100
Sycamore		7000
Spruce, unseasoned		6500
Spruce, seasoned		6800

STONE AND CEMENTS.	Mean-tons per sq. foot.
Limestone	625
Sandstone	425
Brick	175
Ordinary crack	25
In cement	35
First-class cement	60
Concrete	40
Portland cement	120

LEAST WIDTH OF SQUARE PINE POSTS, IN INCHES.

Breaking Load in Tons.

Height.	3	4	5	6	7	8	9	10	11	12	13	14	15	16
FEET.														
4	9.9	22.7	40.8	63.8	92.1	125.0	163.0	204.0	251.0	302.0	358.0	418.0	482.0	552.0
6	6.1	15.6	30.5	51.0	76.8	108.0	143.0	184.0	231.0	281.0	335.0	394.0	456.0	526.0
8	3.9	10.8	12.5	39.7	62.4	90.8	124.0	163.0	207.0	255.0	303.0	367.0	429.0	500.0
10	2.7	7.8	16.9	30.9	50.3	75.3	106.0	142.0	183.0	230.0	281.0	339.0	400.0	466.0
12	2.0	5.8	12.9	24.3	40.6	62.5	87.0	122.0	160.0	204.0	252.0	307.0	365.0	432.0
14	1.4	4.4	10.1	19.4	33.1	51.9	76.0	105.0	140.0	180.0	225.0	277.0	333.0	397.0
16	1.1	3.5	8.1	15.8	27.3	43.4	64.0	90.0	122.0	159.0	201.0	250.0	303.0	363.0
18	0.92	2.8	6.6	13.0	22.7	36.6	55.0	78.0	106.0	140.0	179.0	224.0	274.0	331.0
20	0.76	2.3	5.5	10.9	19.2	31.1	47.0	68.0	93.0	124.0	160.0	201.0	248.0	301.0
22			4.6	9.2	16.3	26.8	40.0	59.0	82.0	109.0	143.0	182.0	224.0	274.0
24			3.9	7.9	14.1	23.2	36.0	52.0	72.0	97.0	127.0	163.0	203.0	249.0
26			3.4	6.8	12.2	20.1	30.0	49.0	64.0	87.0	115.0	148.0	184.0	226.0
28			2.9	5.9	10.7	17.7	28.0	41.0	57.0	78.0	103.0	133.0	167.0	206.0
30			2.1	5.2	9.4	15.3	25.0	34.0	51.0	70.0	93.0	121.0	152.0	189.0
FEET.														
Height.	3	4	5	6	7	8	9	10	11	12	13	14	15	16

STRENGTH OF TIMBER POSTS.

Formula for the ultimate strength of square or rectangular posts of moderately seasoned white and yellow pine, with ends flat and fixed:

$$P = \frac{f}{1 + \frac{1}{250}\left(\frac{l}{h}\right)^2}$$

Where P = crushing load per square inch.
f = 5000 pounds per square inch.
l = length of post, in inches.
h = least width of post, in inches.
$\frac{l}{h}$ = ratio of length to least width.

WOODEN BEAMS AND GIRDERS.

From the general equation $M_o = q\,f\,h\,S$ we can determine the carrying capacity of wooden beams. Now for rectangular sections, q being equal to $2\left(\frac{r^2}{h^2}\right)$, becomes $q = \frac{1}{6}$, since r^2, for rectangular section, is $\frac{h^2}{12}$; whence the general expression becomes

$$M_o = \frac{f\,h\,S}{6} \qquad (1)$$

For beams uniformly loaded over their length, and supported at the ends,

$$M_o = \frac{Wl}{8} \qquad (2)$$

where W is the *total* load on beam and l is the span; and this must be equated to second member of (1). Thus

$$\frac{Wl}{8} = \frac{fhS}{6} \tag{3}$$

For beams of seasoned white pine for building purposes we may take f, the extreme fibre stress, as 1200 pounds per square inch; then

$$\frac{Wl}{8} = \frac{1200}{6} hS \tag{4}$$

whence

$$Wl = 1600\,hS \tag{5}$$

If l be taken in feet, and h in inches, and S, the area, in square inches, then (5) becomes

$$W.l' = \frac{400}{3} hS \tag{6}$$

that is,

$$W = \frac{400}{3} \cdot \frac{hS}{l'} \tag{7}$$

that is, the uniformly distributed total load in pounds which a beam can safely carry is (the height in inches multiplied by the area of beam in square inches, and divided by the span in feet) $\times$ the factor $\frac{400}{3}$.

Now the area $S = bh$; whence (7) becomes

$$W = \frac{400}{3} \quad \frac{bh^2}{l'} \tag{8}$$

If in this, (8), the breadth be taken as 1″,

$$W = \frac{400}{3} \quad \frac{h^2}{l'} \tag{9}$$

We give a table of carrying capacities of 1″ broad white pine beams of varying depths for varying spans. For any beam whose width is b inches, merely multiplying the tabular number by such breadth b, and we get the capacity for the beam in question.

TABLE OF SAFE CARRYING CAPACITY, IN LBS., FOR SEASONED WHITE PINE BEAMS.

1″ in width, the load being uniformly distributed, and the extreme fibre stress 1200 pounds per square inch.

Span, in feet, varying by 2 feet.	DEPTH, IN INCHES.										
	4	6	8	10	12	14	16	18	20	22	24
6	355	800	1422	2222	3200	4355	5688	7200	8888		
8	266	600	1066	1666	2400	3266	4266	5400	6666	8069	
10	213	480	853	1333	1920	2613	3413	4320	5333	6453	7680
12	177	400	711	1111	1600	2177	2844	3600	4444	5377	6400
14	152	343	609	952	1371	1866	2438	3085	3809	4609	5484
16	133	300	533	833	1200	1633	2133	2700	3333	4033	4800
18	118	266	474	740	1066	1452	1896	2400	2962	3585	4264
20	106	240	427	666	960	1306	1706	2160	2666	3226	3840
22		218	388	606	873	1188	1551	1962	2424	2933	3492
24		200	355	555	800	1088	1422	1800	2222	2689	3200
26			328	513	738	1005	1313	1661	2051	2482	2952
28			304	476	685	933	1219	1542	1905	2304	2740
30			284	444	640	871	1138	1440	1777	2151	2560

SHEARING AND BEARING VALUE OF RIVETS.

Diameter of rivet.	Area of rivet.	Single shear, at 3.0 tons per sq. inch.	Bearing value for different thicknesses of plate, at 6.0 tons per square inch. Diameter of rivet × thickness of plate × 6.0 tons.												
			$\frac{1}{4}''$	$\frac{5}{16}''$	$\frac{3}{8}''$	$\frac{7}{16}''$	$\frac{1}{2}''$	$\frac{9}{16}''$	$\frac{5}{8}''$	$\frac{11}{16}''$	$\frac{3}{4}''$	$\frac{13}{16}''$	$\frac{7}{8}''$	$\frac{15}{16}''$	$1''$
$\frac{3}{8}$	0.1104	0.331	0.562	0.703	0.844										
$\frac{7}{16}$	0.1503	0.451	0.656	0.820	0.984	1.148									
$\frac{1}{2}$	0.1963	0.589	0.750	0.937	1.125	1.313	1.500								
$\frac{9}{16}$	0.2485	0.745	0.844	1.055	1.266	1.475	1.687	1.900							
$\frac{5}{8}$	0.3068	0.920	0.937	1.172	1.417	1.640	1.875	2.110	2.344						
$\frac{11}{16}$	0.3712	1.114	1.031	1.290	1.547	1.805	2.062	2.320	2.578	2.836					
$\frac{3}{4}$	0.4418	1.325	1.125	1.406	1.687	1.968	2.249	2.530	2.811	3.092	3.375				
$\frac{13}{16}$	0.5185	1.555	1.219	1.524	1.828	2.134	2.438	2.743	3.047	3.352	3.655	3.960			
$\frac{7}{8}$	0.6013	1.804	1.312	1.640	1.968	2.296	2.624	2.952	3.280	3.608	3.936	4.264	4.592		
$\frac{15}{16}$	0.6903	2.071	1.406	1.757	2.110	2.462	2.813	3.165	3.516	3.867	4.219	4.570	4.921	5.273	
1	0.7854	2.356	1.500	1.875	2.250	2.625	3.000	3.375	3.750	4.125	4.500	4.875	5.250	5.625	6.000
$1\frac{1}{16}$	0.8866	2.660	1.594	1.992	2.391	2.789	3.188	3.586	3.985	4.383	4.782	5.180	5.579	5.977	6.375
$1\frac{1}{8}$	0.9940	2.972	1.688	2.109	2.531	2.953	3.375	3.797	4.219	4.640	5.062	5.485	5.907	6.329	6.750
$1\frac{3}{16}$	1.1075	3.322	1.782	2.227	2.672	3.117	3.562	4.007	4.453	4.898	5.344	5.789	6.235	6.680	7.125

BEARING VALUES AND MOMENTS OF RESISTANCE OF PINS.

Diameter of pin, d, in inches.	Area of pin, S, in square inches.	Diameter × area, d S.	Bearing value for 1″ thickness of bearing.		Moments of resistance for fibre stresses of $\left(M_0 = f \frac{d\,S}{8} = \text{inch-tons.}\right)$				
			6.0 tons per sq. in.	7.5 tons per sq. in.	7.5 tons per sq. in.	8.0 tons per sq. in.	9.0 tons per sq. in.	10.0 tons per sq. in.	12.5 tons per sq. in.
			Values, in tons.		Values, in inch-tons.				
2	3.142	6.28	12.00	15.00	5.89	6.28	7.07	7.85	9.81
2⅛	3.546	7.54	12.75	15.94	7.07	7.54	8.48	9.42	11.78
2¼	3.976	8.95	13.50	16.88	8.39	8.95	10.07	11.19	13.99
2⅜	4.430	10.52	14.25	17.81	9.86	10.52	11.84	13.15	16.44
2½	4.909	12.27	15.00	18.75	11.50	12.27	13.81	15.34	19.18
2⅝	5.412	14.21	15.75	19.69	13.32	14.21	15.98	17.76	22.20
2¾	5.940	16.34	16.50	20.63	15.32	16.34	18.38	20.42	25.53
2⅞	6.492	18.66	17.25	21.56	17.49	18.66	20.99	23.32	29.15
3	7.069	21.21	18.00	22.50	19.88	21.21	23.86	26.51	33.14
3⅛	7.670	23.97	18.75	23.44	22.47	23.97	26.96	29.96	37.45
3¼	8.296	26.96	19.50	24.38	25.28	26.96	30.33	33.70	42.12
3⅜	8.946	30.19	20.25	25.31	28.30	30.19	33.97	37.74	47.18
3½	9.621	33.67	21.00	26.25	31.57	33.67	37.88	42.09	52.61
3⅝	10.321	37.41	21.75	27.19	35.09	37.41	42.11	46.79	58.49
3¾	11.045	41.42	22.50	28.13	38.83	41.42	46.59	51.77	64.71
3⅞	11.793	45.70	23.25	29.06	42.84	45.70	51.41	57.12	71.40
4	12.566	50.26	24.00	30.00	47.11	50.26	56.54	62.82	78.52
4⅛	13.364	55.13	24.75	30.94	51.68	55.13	62.02	68.91	86.14
4¼	14.186	60.29	25.50	31.88	56.52	60.29	67.82	75.36	94.20
4⅜	15.033	65.77	26.25	32.81	61.66	65.77	73.99	82.21	102.76
4½	15.904	71.57	27.00	33.75	67.09	71.57	80.51	89.46	111.83
4⅝	16.800	77.70	27.75	34.69	72.84	77.70	87.41	97.12	121.40
4¾	17.721	84.18	28.50	35.63	78.92	84.18	94.70	105.22	131.52
4⅞	18.665	90.99	29.25	36.56	85.30	90.99	102.37	113.74	142.18
5	19.635	98.18	30.00	37.50	92.04	98.18	110.45	122.72	153.40
5⅛	20.629	105.72	30.75	38.44	99.11	105.72	118.94	132.15	165.19
5¼	21.648	113.65	31.50	39.38	106.55	113.65	127.85	142.06	177.58
5⅜	22.691	121.96	32.25	40.31	114.34	121.96	137.21	152.45	190.56
5½	23.758	130.67	33.00	41.25	122.51	130.67	147.00	163.34	204.18
5⅝	24.850	139.78	33.75	42.19	131.04	139.78	157.25	174.72	218.40
5¾	25.967	149.31	34.50	43.13	139.98	149.31	167.98	186.64	233.30
5⅞	27.109	159.26	35.25	44.06	149.31	159.26	179.17	199.08	248.85
6	28.274	169.64	36.00	45.00	159.04	169.64	190.85	212.05	265.06
6⅛	29.465	180.47	36.75	45.94	169.19	180.47	203.03	225.59	281.99
6¼	30.680	191.75	37.50	46.88	179.77	191.75	215.72	239.69	299.61
6⅜	31.919	203.48	38.25	47.81	190.76	203.48	228.92	254.35	317.94
6½	33.183	215.69	39.00	48.75	202.21	215.69	242.65	269.61	337.02
6⅝	34.472	228.38	39.75	49.68	214.10	228.38	256.92	285.47	356.84
6¾	35.785	241.55	40.50	50.63	226.45	241.55	271.75	301.94	377.42
6⅞	37.122	255.21	41.25	51.56	239.26	255.21	287.11	319.01	398.76
7	38.485	269.40	42.00	52.50	252.56	269.40	303.08	336.75	420.94
7¼	41.282	299.29	43.50	54.38	280.58	299.29	336.70	374.11	467.64
7½	44.179	331.34	45.00	56.25	310.63	331.34	372.76	414.18	517.73
7¾	47.173	365.60	46.50	58.13	342.75	365.60	411.30	457.00	571.25
8	50.265	402.12	48.00	60.00	376.99	402.12	452.39	502.65	628.31

WIND PRESSURE

Upon the inclined surfaces of roofs.

If $P =$ intensity of wind pressure in pounds per square foot upon any surface normal to its direction, and $\phi =$ angle made by roof surface with the direction of wind, then the normal pressure on the roof surface is given by

$$P_n = P. \sin \phi.^{1.84 \cos \phi - 1.}$$

Let P_h, P_v, be the components of this normal force P_n, parallel and perpendicular respectively, to the direction of wind; then

$$P_h = P_n. \sin \phi, \text{ and } P_v = P_n. \cos \phi.$$

If P be assumed to blow horizontally, then ϕ is angle made by roof surface with the horizontal, and P_n is perpendicular to roof surface, and P_h and P_v are respectively parallel and perpendicular to direction of wind,—that is, respectively horizontal and vertical wind forces.

TABLE OF NORMAL PRESSURES

And vertical and horizontal components for varying inclinations of roof surface to direction of wind, when P = 40 pounds.

Angle of roof.	Pounds per square foot of surface.		
	P_n	P_v	P_h
5°	5.0	4.9	0.4
10°	9.7	9.6	1.7
20°	18.1	17.0	6.2
30°	26.4	22.8	13.2
40°	33.3	25.5	21.4
50°	38.1	24.5	29.2
60°	40.0	20.0	34.0
70°	41.0	14.0	38.5

TABLE OF MULTIPLIERS

For any wind intensity p pounds per square foot.

Angle of roof, ϕ.	5°.	10°.	20°.	30°.	40°.	50°.	60°.
$p_n = p$ (the wind unit) $\times$	0.125	0.24	0.45	0.66	0.83	0.95	1.00
$p_v = p$ (the wind unit) $\times$	0.122	0.24	0.42	0.57	0.64	0.61	0.50
$p_h = p$ (the wind unit) $\times$	0.010	0.04	0.15	0.33	0.53	0.73	0.85

Thus, for instance, if the angle of roof to the horizontal be 20°, and the wind be assumed as blowing horizontally, we find, from preceding table, that the force of wind normal to roof surface is 18.1 pounds per square foot, the horizontal and vertical components of which are respectively 17.0 pounds per square foot and 6.2 pounds per square foot.

The horizontal component tends to turn the roof framing about the leeward side considered as a fulcrum, and also to slide it off the walls; the vertical component acts as a one-sided load on the windward side of roof trusses. The trusses and framing should be proportioned to resist these eccentric loadings, and not for a *uniform* load distributed over *whole* surface of roof.

Usually, the computation of the stresses is most quickly done by means of the Graphical method.

WEIGHT OF ROOF COVERINGS

In pounds per square foot.

	Lbs.
Slate, $\frac{3}{16}''$ thick, on 1″ boards	10.0
Slate, $\frac{1}{8}''$ thick, on 1″ boards	7.5
Corrugated iron, No. 20, on 1″ boards	6.0
Felt on boards, 3 ply, on 1″ boards =	3.5
Tin on 1″ boards	4.0

	Lbs.
Slate on T purlins	12.0
Corrugated iron and laths	6.0
Slate or iron laths	10.0

When slate is used on purlins of T irons, the purlins should be $2 \times 2 \times \frac{1}{4}$, 10 pounds per yard, and spaced from 10″ to 12″ apart, the spacing between rafters (jacks and principals, or between jacks and jacks) should be about 5 feet.

ANGLES OF ROOFS.

Proportion of rise to span.	Angle.	Slope.	Proportion of rise to span.	Angle.	Slope.
1/6	18° 25′	3 to 1	2/3	53° 00′	3/4 to 1
1/4	26° 35′	2 to 1	3/4	56° 20′	2/3 to 1
1/3	33° 42′	1 1/2 to 1	1	63° 30′	1/2 to 1
1/2	45° 00′	1 to 1			

TABLES OF WEIGHTS

COMPILED FROM VARIOUS SOURCES.

WEIGHT OF BAR IRON.

Size, in inches.	Square bar, 1 foot long.	Round bar, 1 foot long.	Area, in □ inches.	Area, in ○ inches.
$\frac{1}{16}$	0.0132	0.0104	0.0039	0.0031
$\frac{1}{8}$	0.0526	0.0414	0.0156	0.0123
$\frac{3}{16}$	0.1184	0.0930	0.0351	0.0276
$\frac{1}{4}$	0.2105	0.1653	0.0625	0.0491
$\frac{5}{16}$	0.3290	0.2583	0.0976	0.0767
$\frac{3}{8}$	0.4736	0.3720	0.1406	0.1104
$\frac{7}{16}$	0.6446	0.5063	0.1914	0.1503
$\frac{1}{2}$	0.8420	0.6612	0.2500	0.1963
$\frac{9}{16}$	1.0660	0.8370	0.3166	0.2485
$\frac{5}{8}$	1.3160	1.0330	0.3906	0.3068
$\frac{11}{16}$	1.5920	1.2500	0.4727	0.3712
$\frac{3}{4}$	1.8950	1.4880	0.5625	0.4418
$\frac{13}{16}$	2.2230	1.7460	0.6603	0.5185
$\frac{7}{8}$	2.5790	2.0250	0.7656	0.6013
$\frac{15}{16}$	2.9600	2.3250	0.8790	0.6903
1	3.3680	2.6450	1.0000	0.7854
$\frac{1}{16}$	3.8030	2.9860	1.1290	0.8868
$\frac{1}{8}$	4.2630	3.3480	1.2660	0.9940
$\frac{3}{16}$	4.7500	3.7270	1.4090	1.1070
$\frac{1}{4}$	5.2630	4.1330	1.5620	1.2270
$\frac{5}{16}$	5.8020	4.5570	1.7230	1.3530
$\frac{3}{8}$	6.3680	5.0010	1.8910	1.4850
$\frac{7}{16}$	6.9600	5.4660	2.0670	1.6230
$\frac{1}{2}$	7.5780	5.9520	2.2500	1.7670
$\frac{9}{16}$	8.2230	6.4530	2.4390	1.9160
$\frac{5}{8}$	8.8970	6.9850	2.6410	2.0740
$\frac{11}{16}$	9.6460	7.5780	2.8640	2.2500
$\frac{3}{4}$	10.3100	8.1010	3.0630	2.4050
$\frac{13}{16}$	11.0700	8.6930	3.2870	2.5810
$\frac{7}{8}$	11.8400	9.3000	3.5160	2.7610
$\frac{15}{16}$	12.6400	9.9300	3.7520	2.9480
2	13.4700	10.5800	4.0000	3.1420
$\frac{1}{8}$	15.2100	11.9500	4.5160	3.5460
$\frac{1}{4}$	17.0500	13.2900	5.0620	3.9760
$\frac{3}{8}$	19.0000	14.9200	5.6400	4.4300
$\frac{1}{2}$	21.0500	16.5300	6.2500	4.9080
$\frac{5}{8}$	23.2100	18.2300	6.8890	5.4120
$\frac{3}{4}$	25.4700	20.0100	7.5600	5.9390
$\frac{7}{8}$	27.8400	21.8700	8.2640	6.4920
3	30.3100	23.8100	9.0000	7.0690
$\frac{1}{8}$	32.8900	25.8300	9.7640	7.6700
$\frac{1}{4}$	35.5700	27.9400	10.5610	8.2960
$\frac{3}{8}$	38.3600	30.1300	11.3880	8.9460
$\frac{1}{2}$	41.2600	32.4100	12.2500	9.6210

WEIGHT OF BAR IRON.

Size, in inches.	Square bar, 1 foot long.	Round bar, 1 foot long.	Area, in □ inches.	Area, in ○ inches.
5/8	44.250	34.760	13.138	10.321
3/4	47.370	37.200	14.065	11.045
7/8	50.550	39.720	15.010	11.793
4	53.890	42.330	16.000	12.566
1/8	57.290	45.010	17.012	13.364
1/4	60.820	47.780	18.058	14.186
3/8	64.470	50.630	19.141	15.033
1/2	68.210	53.570	20.254	15.904
5/8	72.030	56.590	21.385	16.800
3/4	75.990	59.690	22.556	17.721
7/8	80.000	62.830	23.748	18.655
5	84.200	66.130	25.000	19.635
1/8	88.440	69.480	26.260	20.629
1/4	92.810	72.910	27.557	21.648
3/8	97.280	76.430	28.884	22.690
1/2	101.900	80.020	30.250	23.758
5/8	106.600	83.700	31.641	24.851
3/4	111.400	87.460	33.060	25.967
7/8	116.300	91.310	34.516	27.109
6	121.300	95.230	36.000	28.274
1/4	131.600	103.300	39.063	30.679
1/2	142.300	111.800	42.250	33.183
3/4	153.500	120.500	45.562	35.785
7	165.000	129.600	49.000	38.485
1/4	177.000	139.000	52.562	41.282
1/2	189.500	148.800	56.250	44.179
3/4	202.300	158.900	60.062	47.173
8	215.600	169.300	64.000	50.266
1/4	229.300	180.100	68.062	53.456
1/2	243.400	191.100	72.250	56.745
3/4	247.900	202.500	76.562	60.132
9	272.800	214.300	81.000	63.617
1/4	288.200	226.300	85.563	67.201
1/2	304.000	238.700	90.250	70.882
3/4	320.200	251.500	95.062	74.662
10	336.800	264.500	99.800	78.540
1/4	353.900	277.900	105.400	82.516
1/2	371.300	291.600	110.230	86.590
3/4	389.200	305.700	115.550	90.763
11	407.500	320.100	121.000	95.033
1/4	426.300	334.800	126.540	99.402
1/2	445.400	349.800	132.220	103.870
3/4	465.000	365.200	138.060	108.430
12	485.000	380.900	144.000	113.100

TABLE GIVING DIMENSIONS OF UPSET ENDS

And weights of clevises and sleeve nuts for round and square bars.

ROUND BARS.

Bar.			Upset ends.			Weight of clevises and sleeve nuts for upset ends.	
Diameter of round, in inches.	Weight per foot, in lbs.	Area.	Diameter of upset, in inches.	Length of upset, in inches.	Iron required to make upset, in inches.	One clevise, in lbs.	One sleeve, in lbs.
$\frac{3}{4}$	1.50	0.441	1	4	$3\frac{1}{2}$	$5\frac{1}{2}$	5
$\frac{7}{8}$	2.00	0.601	$1\frac{1}{8}$	4	3	$6\frac{1}{2}$	5
1	2.65	0.785	$1\frac{1}{4}$	4	$2\frac{1}{2}$	$6\frac{1}{2}$	8
$1\frac{1}{8}$	3.35	0.994	$1\frac{3}{8}$	4	$2\frac{1}{4}$	$7\frac{1}{4}$	8
$1\frac{1}{4}$	4.13	1.227	$1\frac{1}{2}$	4 and 6	2 and 3	$7\frac{1}{4}$	9
$1\frac{3}{8}$	5.00	1.484	$1\frac{5}{8}$	4 and 6	$1\frac{3}{4}$ and $2\frac{3}{4}$	9	12
$1\frac{1}{2}$	6.00	1.767	$1\frac{7}{8}$	4 and 6	$2\frac{1}{2}$ and $3\frac{3}{4}$	$13\frac{3}{4}$	13
$1\frac{5}{8}$	7.00	2.073	2	4 and 6	$2\frac{1}{4}$ and $3\frac{1}{2}$	$13\frac{3}{4}$	13
$1\frac{3}{4}$	8.10	2.405	$2\frac{1}{8}$	4 and 6	$2\frac{1}{4}$ and $3\frac{1}{4}$	$20\frac{1}{4}$	16
$1\frac{7}{8}$	9.30	2.761	$2\frac{1}{4}$	6	3	$20\frac{1}{4}$	16
2	10.60	3.141	$2\frac{3}{8}$	6	$2\frac{3}{4}$	$25\frac{1}{4}$	18
$2\frac{1}{8}$	12.00	3.546	$2\frac{1}{2}$	6	$2\frac{1}{2}$	$25\frac{1}{4}$	18
$2\frac{1}{4}$	13.30	3.976	$2\frac{5}{8}$	6	$2\frac{1}{2}$		25
$2\frac{3}{8}$	15.00	4.430	$2\frac{3}{4}$	6	$2\frac{1}{4}$		25
$2\frac{1}{2}$	16.50	4.908	$2\frac{7}{8}$	6	$2\frac{1}{4}$		30
$2\frac{5}{8}$	18.20	5.411	3	6	2		30

SQUARE BARS.

Bar.			Upset ends.			Weight of clevises and sleeve nuts for upset ends.	
Size of bar, in inches.	Weight per foot, in lbs.	Area.	Diameter of upset, in inches.	Length of upset, in inches.	Iron required to make upset, in inches.	One clevise, in lbs.	One sleeve, in lbs.
$\frac{3}{4}$	1.89	0.5625	$1\frac{1}{8}$	4	4	$6\frac{1}{2}$	5
$\frac{7}{8}$	2.57	0.7656	$1\frac{3}{8}$	4	5	$7\frac{1}{4}$	8
1	3.36	1.000	$1\frac{1}{2}$	4	4	$7\frac{1}{4}$	9
$1\frac{1}{8}$	4.26	1.266	$1\frac{5}{8}$	4	4	9	12
$1\frac{1}{4}$	5.26	1.562	$1\frac{7}{8}$	4	4	$13\frac{3}{4}$	13
$1\frac{5}{16}$	5.80	1.725	2	4	4	$13\frac{3}{4}$	13
$1\frac{3}{8}$	6.36	1.891	2	4	4	$13\frac{3}{4}$	13
$1\frac{7}{16}$	6.96	2.067	$2\frac{1}{8}$	$4\frac{1}{2}$	4	$20\frac{1}{4}$	16
$1\frac{1}{2}$	7.57	2.250	$2\frac{1}{4}$	$4\frac{1}{2}$	4	$20\frac{1}{4}$	16
$1\frac{9}{16}$	8.22	2.439	$2\frac{3}{8}$	$4\frac{1}{2}$	4	$25\frac{1}{4}$	18
$1\frac{5}{8}$	8.89	2.641	$2\frac{3}{8}$	$4\frac{1}{2}$	4	$25\frac{1}{4}$	18
$1\frac{11}{16}$	9.64	2.864	$2\frac{1}{2}$	$4\frac{3}{4}$	$3\frac{3}{4}$	$25\frac{1}{4}$	18
$1\frac{3}{4}$	10.31	3.063	$2\frac{5}{8}$	$4\frac{3}{4}$	4		25
$1\frac{13}{16}$	11.07	3.287	$2\frac{5}{8}$	$4\frac{3}{4}$	$3\frac{1}{2}$		25
$1\frac{7}{8}$	11.84	3.516	$2\frac{3}{4}$	5	$3\frac{3}{4}$		25
$1\frac{15}{16}$	12.64	3.752	$2\frac{3}{4}$	5	$3\frac{1}{4}$		25
2	13.47	4.000	$2\frac{7}{8}$	5	$3\frac{1}{2}$		30

WEIGHT OF WROUGHT-IRON BARS.

Width.	THICKNESS, IN INCHES.													Width.
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	
1	0.84	1.05	1.26	1.47	1.68	1.89	2.11	2.32	2.53	2.74	2.95	3.16	3.37	1
1 1/8	0.95	1.18	1.42	1.66	1.90	2.14	2.37	2.61	2.84	3.08	3.32	3.56	3.79	1 1/8
1 1/4	1.05	1.32	1.58	1.84	2.11	2.37	2.63	2.89	3.16	3.42	3.68	3.94	4.21	1 1/4
1 3/8	1.16	1.45	1.74	2.03	2.32	2.61	2.89	3.18	3.47	3.76	4.05	4.34	4.63	1 3/8
1 1/2	1.26	1.58	1.90	2.21	2.53	2.85	3.16	3.48	3.79	4.11	4.42	4.74	5.05	1 1/2
1 5/8	1.37	1.71	2.05	2.39	2.74	3.08	3.42	3.76	4.11	4.45	4.79	5.13	5.47	1 5/8
1 3/4	1.47	1.84	2.21	2.58	2.95	3.32	3.68	4.05	4.42	4.79	5.16	5.53	5.89	1 3/4
1 7/8	1.58	1.97	2.37	2.76	3.16	3.56	3.95	4.35	4.74	5.14	5.53	5.93	6.32	1 7/8
2	1.68	2.11	2.53	2.95	3.37	3.79	4.21	4.63	5.05	5.47	5.89	6.31	6.74	2
2 1/4	1.90	2.37	2.84	3.32	3.79	4.26	4.74	5.21	5.68	6.15	6.83	7.30	7.58	2 1/4
2 1/2	2.11	2.63	3.16	3.68	4.21	4.74	5.26	5.79	6.32	6.85	7.37	7.90	8.42	2 1/2
2 3/4	2.32	2.89	3.47	4.05	4.63	5.21	5.79	6.37	6.95	7.53	8.10	8.68	9.26	2 3/4
3	2.53	3.16	3.79	4.42	5.05	5.68	6.32	6.92	7.58	8.21	8.84	9.47	10.10	3
3 1/4	2.74	3.42	4.11	4.79	5.47	6.15	6.84	7.52	8.22	9.09	9.58	10.26	10.95	3 1/4
3 1/2	2.95	3.68	4.42	5.16	5.89	6.63	7.37	8.11	8.84	9.58	10.32	11.06	11.79	3 1/2
3 3/4	3.16	3.95	4.74	5.53	6.32	7.11	7.89	8.68	9.47	10.26	11.05	11.84	12.63	3 3/4
4	3.37	4.21	5.05	5.89	6.74	7.58	8.42	9.26	10.10	10.94	11.79	12.63	13.47	4
4 1/4	3.58	4.47	5.37	6.26	7.16	8.06	8.95	9.85	10.74	11.64	12.53	13.43	14.31	4 1/4
4 1/2	3.79	4.74	5.68	6.63	7.58	8.53	9.47	10.42	11.38	12.33	13.26	14.21	15.16	4 1/2
4 3/4	4.00	5.00	6.00	7.00	8.00	9.00	10.00	11.00	12.00	13.00	14.00	15.00	16.00	4 3/4
5	4.21	5.26	6.32	7.37	8.42	9.47	10.53	11.58	12.63	13.68	14.74	15.79	16.84	5
5 1/4	4.42	5.53	6.53	7.74	8.84	9.94	11.05	12.16	13.26	14.37	15.47	16.58	17.68	5 1/4
5 1/2	4.63	5.79	6.95	8.10	9.26	10.42	11.58	12.74	13.89	15.05	16.21	17.37	18.52	5 1/2
5 3/4	4.84	6.05	7.26	8.47	9.68	10.89	12.10	13.31	14.53	15.74	16.95	18.16	19.37	5 3/4

WEIGHT OF WROUGHT-IRON FLATS.

THICKNESS, IN INCHES.

Width.	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	Width.
6	5.05	6.32	7.58	8.84	10.10	11.36	12.63	13.89	15.16	16.42	17.68	18.94	20.21	6
6 1/4	5.27	6.58	7.90	9.21	10.53	11.84	13.16	14.47	15.79	17.10	18.42	19.73	21.05	6 1/4
6 1/2	5.47	6.84	8.21	9.58	10.94	12.30	13.68	15.04	16.42	17.78	19.16	20.52	21.88	6 1/2
6 3/4	5.69	7.10	8.53	9.95	11.36	12.78	14.21	15.63	17.05	18.47	19.90	21.32	22.73	6 3/4
7	5.90	7.36	8.84	10.32	11.79	13.36	14.74	16.21	17.68	19.15	20.64	22.11	23.58	7
7 1/4	6.11	7.63	9.16	10.68	12.21	13.74	15.26	16.79	18.32	19.85	21.37	22.90	24.42	7 1/4
7 1/2	6.32	7.90	9.48	11.06	12.64	14.22	15.78	17.36	18.94	20.52	22.11	23.69	25.28	7 1/2
7 3/4	6.53	8.16	9.79	11.42	13.06	14.69	16.31	17.94	19.57	21.20	22.84	24.47	26.12	7 3/4
8	6.74	8.42	10.10	11.78	13.48	15.16	16.84	18.52	20.20	21.88	23.58	25.26	26.94	8
8 1/4	6.95	8.68	10.42	12.16	13.89	15.63	17.37	19.11	20.84	22.58	24.32	26.06	27.79	8 1/4
8 1/2	7.16	8.94	10.74	12.52	14.32	16.11	17.90	19.69	21.48	23.27	25.06	26.85	28.63	8 1/2
8 3/4	7.37	9.21	11.05	12.89	14.74	16.58	18.42	20.26	22.10	23.94	25.79	27.63	29.47	8 3/4
9	7.58	9.48	11.36	13.26	15.16	17.06	18.95	20.85	22.75	24.65	26.52	28.42	30.32	9
9 1/4	7.79	9.74	11.68	13.63	15.58	17.53	19.47	21.42	23.38	25.33	27.26	29.21	31.16	9 1/4
9 1/2	8.00	10.00	12.00	14.00	16.00	18.00	20.00	22.00	24.00	26.00	28.00	30.00	32.00	9 1/2
9 3/4	8.21	10.26	12.32	14.37	16.42	18.47	20.53	22.58	24.63	26.68	28.74	30.79	32.84	9 3/4
10	8.42	10.52	12.64	14.74	16.84	18.94	21.05	23.15	25.26	27.36	29.48	31.58	33.68	10
10 1/4	8.63	10.79	12.95	15.11	17.26	19.42	21.58	23.74	25.89	28.05	30.21	32.37	34.52	10 1/4
10 1/2	8.84	11.05	13.26	15.48	17.68	19.89	22.10	24.31	26.52	28.73	30.95	33.16	35.36	10 1/2
10 3/4	9.05	11.32	13.58	15.84	18.10	20.36	22.63	24.89	27.16	29.42	31.68	33.94	36.21	10 3/4
11	9.26	11.58	13.90	16.21	18.52	20.84	23.16	25.48	27.78	30.10	32.42	34.74	37.04	11
11 1/4	9.47	11.85	14.21	16.58	18.94	21.31	23.68	26.05	28.42	30.79	33.15	35.52	37.89	11 1/4
11 1/2	9.68	12.10	14.52	16.94	19.36	21.78	24.20	26.62	29.06	31.48	33.90	36.32	38.74	11 1/2
11 3/4	9.89	12.37	14.84	17.31	19.78	22.25	24.73	27.20	29.69	32.16	34.63	37.10	39.56	11 3/4
12	10.10	12.64	15.16	17.68	20.20	22.72	25.26	27.78	30.32	32.84	35.36	37.88	40.40	12

WEIGHTS FOR PLATES OVER TWELVE INCHES WIDE.

Width.	THICKNESS, IN INCHES.													Width.
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1	
13	10.94	13.69	16.42	19.15	21.88	24.60	27.36	30.08	32.84	35.56	38.32	41.04	43.76	13
14	11.78	14.72	17.68	20.64	23.58	26.72	29.48	32.42	35.36	38.30	41.28	44.22	47.16	14
15	12.62	15.77	18.94	22.11	25.26	28.61	31.59	34.74	37.89	41.04	44.23	47.38	50.53	15
16	13.46	16.84	20.20	23.56	26.96	30.32	33.68	37.04	40.40	43.76	47.16	50.52	53.88	16
17	14.30	17.89	21.46	25.03	28.64	32.21	35.79	39.36	42.93	46.50	50.11	53.68	57.25	17
18	15.16	18.96	22.72	26.52	30.32	34.12	37.90	41.70	45.50	49.30	53.04	56.84	60.62	18
19	16.00	20.01	23.98	27.99	32.00	36.01	40.01	44.02	48.03	52.04	55.99	60.00	63.99	19
20	16.84	21.04	25.28	29.48	33.68	37.88	42.10	46.30	50.52	54.72	58.96	62.16	67.36	20
21	17.68	22.09	26.54	31.95	35.36	39.77	44.21	48.62	53.05	57.46	61.91	65.32	71.73	21
22	18.52	23.16	27.80	32.42	37.04	41.68	46.32	50.96	55.56	60.20	64.82	69.48	74.08	22
23	19.36	24.21	29.06	33.89	38.72	43.57	48.43	53.28	58.09	62.94	67.77	73.64	77.45	23
24	20.20	25.28	30.32	35.38	40.40	45.44	50.52	55.56	60.64	65.68	70.72	75.76	80.80	24
25	21.04	26.33	31.58	36.85	42.08	47.33	52.63	57.88	63.17	68.42	73.67	78.92	84.17	25
26	21.88	27.39	32.85	38.33	43.77	49.23	54.73	60.19	65.69	71.15	76.61	82.07	87.54	26
27	22.72	28.44	34.11	39.80	45.45	51.12	56.84	62.51	68.22	73.89	79.56	85.23	90.91	27
28	23.56	29.44	35.36	41.28	47.16	53.44	58.96	64.84	70.72	76.60	82.56	88.44	94.32	28
29	24.40	30.49	36.62	42.75	48.84	55.33	61.07	67.16	73.25	79.34	85.51	91.60	97.69	29
30	25.24	31.55	37.89	44.23	50.53	57.23	63.17	69.47	75.77	82.07	88.45	94.75	101.06	30
31	26.08	32.60	39.15	45.70	52.21	59.12	65.28	71.79	78.30	84.81	91.40	97.91	104.43	31
32	26.92	33.68	40.40	47.12	53.92	60.64	67.36	74.08	80.80	87.52	95.32	101.04	107.76	32
33	27.76	34.73	41.66	48.59	55.60	62.53	69.47	76.40	83.33	90.26	98.27	104.20	111.13	33
34	28.60	35.79	42.93	50.07	57.29	64.43	71.57	78.71	85.85	92.99	101.21	107.35	114.50	34
35	29.44	36.84	44.19	51.54	58.97	66.32	73.68	81.03	88.38	95.73	104.16	110.51	117.87	35
36	30.32	37.92	45.44	53.04	60.64	68.24	75.80	83.40	91.00	98.60	106.08	113.68	121.24	36

WEIGHTS FOR PLATES OVER TWELVE INCHES WIDE.

Width.	THICKNESS, IN INCHES.													Width.
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	
37	31.16	38.97	46.70	54.51	62.32	70.13	77.91	85.72	93.53	101.34	109.03	116.84	124.61	37
38	32.00	40.00	48.00	56.00	64.00	72.00	80.00	88.00	96.00	104.00	112.00	120.00	128.00	38
39	32.84	41.05	49.26	57.47	65.68	71.89	82.11	90.32	98.53	106.74	114.95	123.16	131.37	39
40	33.68	42.08	50.56	58.96	67.36	75.76	84.20	92.60	101.04	109.44	117.92	124.32	134.72	40
41	34.52	43.13	51.82	60.43	69.04	77.65	86.31	94.92	103.57	112.18	120.87	127.48	138.09	41
42	35.36	44.19	53.09	61.91	70.73	79.55	88.41	97.23	106.09	114.91	123.81	130.63	141.46	42
43	36.20	45.24	54.35	63.38	72.41	81.44	90.52	99.55	108.62	117.65	126.76	133.79	144.83	43
44	37.04	46.32	55.60	64.84	74.08	83.36	92.64	101.92	111.12	120.40	129.64	138.96	148.08	44
45	37.88	47.37	56.86	66.31	75.76	85.25	94.75	104.24	113.65	123.14	132.59	142.12	151.45	45
46	38.72	48.43	58.13	67.79	77.45	88.15	96.85	106.55	116.17	125.87	135.53	145.27	154.82	46
47	39.56	49.48	59.39	69.26	79.13	89.04	98.96	108.87	118.70	128.61	138.48	148.43	158.19	47
48	40.40	50.56	60.64	70.76	80.80	90.88	101.04	111.12	121.28	131.36	141.44	151.52	161.60	48
49	41.24	51.61	61.90	72.23	82.48	92.77	103.15	113.42	123.81	134.10	144.39	154.68	164.97	49
50	42.08	52.67	63.17	73.71	84.17	94.67	105.25	115.75	126.33	136.83	147.33	157.83	168.34	50
51	42.92	53.73	64.43	75.18	85.85	96.56	107.36	118.07	128.86	139.57	150.28	160.99	171.71	51
52	43.74	54.78	65.70	76.66	87.54	98.46	109.46	120.38	131.38	142.30	153.22	164.14	175.08	52
53	44.58	55.83	66.96	78.13	89.22	100.35	111.57	122.70	133.91	145.04	156.17	167.30	178.45	53
54	45.42	56.89	68.23	79.61	90.91	102.25	113.67	125.01	136.43	147.77	159.11	170.55	181.82	54
55	46.26	57.94	69.49	81.08	92.59	104.14	115.78	127.33	138.96	150.51	162.06	173.71	185.19	55
56	47.12	58.88	70.72	82.56	94.32	106.88	117.92	129.64	141.44	153.20	165.12	176.88	188.64	56
57	47.96	59.93	71.98	84.03	96.00	108.77	120.03	131.96	143.97	155.94	168.07	180.04	192.01	57
58	48.80	60.99	73.25	85.51	97.69	110.67	122.13	134.27	146.49	158.67	171.01	183.19	195.38	58
59	49.64	62.04	74.51	86.98	99.37	112.46	124.24	136.59	149.02	161.41	173.96	186.35	198.75	59
60	50.48	63.10	75.78	88.46	101.06	114.46	126.34	138.90	151.54	164.14	176.90	189.50	202.12	60

WEIGHT OF

BARS OVER ONE INCH IN THICKNESS,

Per lineal foot of length.

Thickness, in inches.	WIDTH, IN INCHES.								Thickness, in inches.
	1	2	3	4	5	6	7	8	
$1\frac{1}{16}$	3.6	7.2	10.7	14.3	17.9	21.5	25.0	28.6	$1\frac{1}{16}$
$1\frac{1}{8}$	3.8	7.6	11.4	15.2	19.0	22.7	26.5	30.4	$1\frac{1}{8}$
$1\frac{3}{16}$	4.0	8.0	12.0	16.0	20.0	24.0	28.0	32.0	$1\frac{3}{16}$
$1\frac{1}{4}$	4.2	8.4	12.6	16.8	21.1	25.3	29.5	33.6	$1\frac{1}{4}$
$1\frac{5}{16}$	4.5	8.9	13.3	17.7	22.1	26.5	31.0	35.4	$1\frac{5}{16}$
$1\frac{3}{8}$	4.7	9.3	13.9	18.5	23.2	27.8	32.4	37.0	$1\frac{3}{8}$
$1\frac{7}{16}$	4.9	9.7	14.5	19.4	24.2	29.1	33.9	38.8	$1\frac{7}{16}$
$1\frac{1}{2}$	5.1	10.1	15.2	20.2	25.3	30.3	35.4	40.4	$1\frac{1}{2}$
$1\frac{9}{16}$	5.3	10.6	15.8	21.1	26.3	31.6	36.9	42.2	$1\frac{9}{16}$
$1\frac{5}{8}$	5.5	10.9	16.4	21.9	27.4	32.8	38.3	43.8	$1\frac{5}{8}$
$1\frac{11}{16}$	5.7	11.4	17.0	22.7	28.4	34.1	39.8	45.4	$1\frac{11}{16}$
$1\frac{3}{4}$	5.9	11.8	17.6	23.6	29.5	35.6	41.3	47.2	$1\frac{3}{4}$
$1\frac{13}{16}$	6.1	12.2	18.3	24.4	30.5	36.6	42.7	48.8	$1\frac{13}{16}$
$1\frac{7}{8}$	6.3	12.6	18.9	25.3	31.5	37.9	44.2	50.6	$1\frac{7}{8}$
$1\frac{15}{16}$	6.5	13.0	19.6	26.1	32.6	39.2	45.7	52.2	$1\frac{15}{16}$
2	6.7	13.4	20.2	26.9	33.7	40.4	47.2	53.8	2

BOLTS, WITH SQUARE HEADS AND NUTS.

Weight of one hundred of the enumerated sizes.

HOOPES & TOWNSEND, PHILADELPHIA.

Length, in inches.	1/4″	3/8″	1/2″	5/8″	3/4″	7/8″	1″	1 1/8″
1 1/2	4.16	10.62	23.87	39.31				
1 3/4	4.22	11.72	25.06	41.38				
2	4.75	12.38	26.44	45.69	73.62			
2 1/4	5.34	12.90	28.62	49.50	76.00			
2 1/2	5.97	14.69	29.50	51.25	79.75			
2 3/4	6.50	16.47	31.16	53.00	83.00			
3		17.87	32.44	56.00	85.38	127.25		
3 1/2		18.94	39.75	63.12	93.44	140.56		
4		20.59	42.50	74.87	108.12	148.37	228.0	296.0
4 1/2		21.69	44.87	79.62	113.12	158.76	239.0	310.0
5		23.62	48.81	83.00	122.00	167.25	250.0	324.0
5 1/2		25.81	51.38	87.88	128.62	174.88	261.0	338.0
6		26.87	53.31	92.38	131.75	204.25	272.0	352.0
6 1/2			56.87	96.88	139.56	214.69	283.0	366.0
7			59.12	99.87	145.50	228.44	294.0	370.0
7 1/2			61.87	105.75	150.88	235.31	305.0	384.0
8			64.44	109.50	157.12	239.88	316.0	398.0
9			70.50	118.12	169.62	258.12	338.0	426.0
10			77.00	128.13	184.00	276.18	360.0	454.0
11			82.88	136.19	195.13	295.69	382.0	482.0
12			86.37	144.87	209.75	311.94	404.0	510.0
13			92.00	155.50	219.37	335.81	426.0	538.0
14			97.75	163.58	237.50	351.88	448.0	566.0
15			103.25	170.75	249.06	391.75	470.0	594.0

Franklin Institute Standard Sizes

SQUARE AND HEXAGON NUTS.

Number of each size in 100 lbs. These nuts are chamfered and trimmed.

HOOPES & TOWNSEND, PHILADELPHIA.

Width.	Thickness.	Hole.	Size of bolt.	No. of square.	No. of hexagon.
1/2	1/4	13/64	1/4	8140	9300
19/32	5/16	1/4	5/16	3000	6200
11/16	3/8	5/16	3/8	2320	3120
25/32	7/16	11/32	7/16	1940	2200
7/8	1/2	13/32	1/2	1180	1350
31/32	9/16	29/64	9/16	920	1000
1 1/16	5/8	33/64	5/8	738	830
1 1/4	3/4	5/8	3/4	420	488
1 7/16	7/8	23/32	7/8	280	309
1 5/8	1	27/32	1	180	216
1 13/16	1 1/8	15/16	1 1/8	130	148
2	1 1/4	1 1/16	1 1/4	96	111
2 3/16	1 3/8	1 5/32	1 3/8	70	85
2 3/8	1 1/2	1 9/32	1 1/2	60	70

WEIGHT OF RIVETS.

Per hundred. Length from under head.

Length, in inches.	DIAMETER, IN INCHES.							
	3/8	1/2	5/8	3/4	7/8	1	1 1/8	1 1/4
1 1/4	5.4	12.6	21.5	28.7	43.1	65.3	91.5	123.0
1 1/2	6.2	13.9	23.7	31.8	47.3	70.7	98.4	133.0
1 3/4	6.9	15.3	25.8	34.9	51.4	76.2	105.0	142.0
2	7.7	16.6	27.9	37.9	55.6	81.6	112.0	150.0
2 1/4	8.5	18.0	30.0	41.0	59.8	87.1	119.0	159.0
2 1/2	9.2	19.4	32.2	44.1	63.0	92.5	126.0	167.0
2 3/4	10.0	20.7	34.3	47.1	68.1	98.0	133.0	176.0
3	10.8	22.1	36.4	50.2	72.3	103.0	140.0	184.0
3 1/4	11.5	23.5	38.6	53.3	76.5	109.0	147.0	193.0
3 1/2	12.3	24.8	40.7	56.4	80.7	114.0	154.0	201.0
3 3/4	13.1	26.2	42.8	59.4	84.8	120.0	161.0	210.0
4	13.8	27.5	45.0	62.5	89.0	125.0	167.0	218.0
4 1/4	14.6	28.9	47.1	65.6	93.2	131.0	174.0	227.0
4 1/2	15.4	30.3	49.2	68.6	97.4	136.0	181.0	236.0
4 3/4	16.2	31.6	51.4	71.7	102.0	142.0	188.0	244.0
5	16.9	33.0	53.5	74.8	106.0	147.0	195.0	253.0
5 1/4	17.7	34.4	55.6	77.8	110.0	153.0	202.0	261.0
5 1/2	18.4	35.7	57.7	80.9	114.0	158.0	209.0	270.0
5 3/4	19.2	37.1	59.9	84.0	118.0	163.0	216.0	278.0
6	20.0	38.5	62.0	87.0	122.0	169.0	223.0	287.0
6 1/2	21.5	41.2	66.3	93.2	131.0	180.0	236.0	304.0
7	23.0	43.9	70.5	99.3	139.0	191.0	250.0	321.0
7 1/2	24.6	46.6	74.8	106.0	147.0	202.0	264.0	338.0
8	26.1	49.4	79.0	112.0	156.0	213.0	278.0	355.0
8 1/2	27.6	52.1	83.3	118.0	164.0	223.0	292.0	372.0
9	29.2	54.8	87.6	124.0	173.0	234.0	306.0	389.0
9 1/2	30.7	57.6	91.8	130.0	181.0	245.0	319.0	406.0
10	32.2	60.3	96.1	136.0	189.0	256.0	333.0	423.0
10 1/2	33.8	63.0	101.0	142.0	198.0	267.0	347.0	440.0
11	35.3	65.7	105.0	148.0	206.0	278.0	361.0	457.0
11 1/2	36.8	68.5	109.0	155.0	214.0	289.0	375.0	474.0
12	38.4	71.2	113.0	161.0	223.0	300.0	388.0	491.0
Heads.	1.8	5.7	10.9	13.4	22.2	38.0	57.0	82.0

Table showing the average weight, in pounds, of one hundred

MACHINE BOLTS

Of various sizes and lengths, having square heads and square nuts.

Lengths	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4	7/8	1
1½	4	6	9¾	15	21	30	35			
2	4¾	7	11	17	24	33½	39	68		
2½	5½	8	12¼	19	26¾	37	43	74	116	
3	6¼	9	14	21	29¾	40½	48	81	124	185
3½	7	10	15½	23	32½	44	52	87	132	196
4	7¾	11	17¼	25	35	47½	56½	93	140	207
4½	8½	12	18½	27	37¾	51	61	100	149	218
5	9¼	13¼	20	29	40	54½	65	106	158	229
5½	10	14¼	21½	31	42¾	58	69	112	166	240
6	10¾	15½	23¼	33	45½	61½	74	118	174	251
6½	11½	16½	25	35	48½	65	78½	125	182	262
7	12¼	17½	26¾	37	51¼	68½	82½	131	190	273
7½	13	18¾	28½	39	53¾	72	87	137	198	284
8	13¾	20	30¼	41	56	75½	91	143	207	295
9			34	45	61½	82½	100	155	223	317
10			37½	49	67	89½	109	168	240	339
11			41	53	72½	96½	118	180	256	360
12			44½	57	78	103½	127	192	272	382
13					83½	110½	135	205	289	404
14					89	117½	144	217	306	426
15					94½	124½	153	230	323	448
16					100	131½	162	242	340	470
17					105½	138½	171	255	357	492
18					111	145½	179	267	374	514
19					116½	152½	188	280	391	536
20					122	159½	197	292	408	558

Sizes and weights of

SQUARE AND HEXAGON NUTS.

	Franklin Institute Standard Sizes.				Hoopes & Townsend's Regular Sizes.					
			Square.	Hexagon.	Square.			Hexagon.		
Size of bolt.	Width.	Thickness	Weight, each, in lbs.	Weight, each, in lbs.	Width.	Thickness.	Weight, each, in lbs.	Width.	Thickness.	Weight, each, in lbs.
1/4	1/2	1/4	0.012	0.011	1/2	1/4	0.015	1/2	1/4	0.012
5/16	19/32	5/16	0.033	0.016	5/8	5/16	0.028	5/8	5/16	0.023
3/8	11/16	3/8	0.043	0.032	3/4	3/8	0.049	3/4	3/8	0.040
7/16	25/32	7/16	0.052	0.045	7/8	3/8	0.072	3/4	7/16	0.046
1/2	7/8	1/2	0.085	0.074	1	1/2	0.119	1	9/16	0.111
9/16	31/32	9/16	0.109	0.100	1 1/8	1/2	0.154	1	9/16	0.114
5/8	1 1/16	5/8	0.135	0.120	1 1/4	5/8	0.244	1 1/8	3/4	0.187
3/4	1 1/4	3/4	0.238	0.205	1 3/8	3/4	0.370	1 3/8	7/8	0.339
7/8	1 7/16	7/8	0.357	0.32	1 1/2	7/8	0.465	1 1/2	1	0.446
1	1 5/8	1	0.556	0.46	1 3/4	1	0.714	1 3/4	1 1/8	0.667
1 1/8	1 13/16	1 1/8	0.769	0.68	2	1 1/8	1.05	2	1 3/8	1.00
1 1/4	2	1 1/4	1.04	0.90	2 1/4	1 1/4	1.39	2	1 3/8	1.04
1 3/8	2 3/16	1 3/8	1.43	1.18	2 1/2	1 1/2	2.22	2 1/4	1 1/2	1.39
1 1/2	2 3/8	1 1/2	1.67	1.43	3	1 1/2	3.12	2 3/4	1 5/8	2.33
1 3/4								3 1/4	1 7/8	3.50
2								3 1/2	2	5.25
2 1/4								3 3/4	2 1/4	5.75
2 1/2								4 1/4	2 3/4	7.25
2 3/4								4 1/2	3	10.0
3								4 3/4	3 1/4	12.0

Standard sizes of

WROUGHT-IRON WASHERS.

Number in 100 pounds.

Diameter, in inches.	Size of hole, in inches.	Thickness of wire gauge. Number.	Size of bolt, in inches.	Number in 100 lbs.
$\frac{5}{8}$	$\frac{5}{16}$	16	$\frac{1}{4}$	29,300
$\frac{3}{4}$	$\frac{3}{8}$	16	$\frac{5}{16}$	18,000
1	$\frac{7}{16}$	14	$\frac{3}{8}$	7,600
$1\frac{1}{2}$	$\frac{9}{16}$	11	$\frac{1}{2}$	3,300
$1\frac{1}{2}$	$\frac{5}{8}$	11	$\frac{9}{16}$	2,180
$1\frac{1}{2}$	$\frac{11}{16}$	11	$\frac{5}{8}$	2,350
$1\frac{3}{4}$	$\frac{13}{16}$	11	$\frac{3}{4}$	1,680
2	$\frac{31}{32}$	10	$\frac{7}{8}$	1,140
$2\frac{1}{2}$	$1\frac{1}{8}$	8	1	580
$2\frac{3}{4}$	$1\frac{1}{4}$	8	$1\frac{1}{8}$	470
3	$1\frac{3}{8}$	7	$1\frac{1}{4}$	360
3	$1\frac{1}{2}$	6	$1\frac{3}{8}$	360

CAST HEADS AND WASHERS,

For combination bolts.

Diameter of bolt, in inches.	Diameter of head or washer, in inches.	Weight of head, in lbs.	Weight of washer, in lbs.	Diameter of bolt, in inches.	Diameter of head or washer, in inches.	Weight of head, in lbs.	Weight of washer, in lbs.
$\frac{1}{2}$	$2\frac{1}{2}$	0.32	0.32	$1\frac{5}{8}$	$6\frac{3}{4}$	7.0	7.0
$\frac{5}{8}$	3	0.67	0.61	$1\frac{3}{4}$	$7\frac{1}{4}$	8.3	8.3
$\frac{3}{4}$	$3\frac{1}{2}$	0.91	0.78	$1\frac{7}{8}$	$7\frac{3}{4}$	10.4	10.4
$\frac{7}{8}$	$3\frac{3}{4}$	0.95	0.89	2	$8\frac{1}{4}$	12.4	12.4
1	$4\frac{1}{4}$	1.7	1.75	$2\frac{1}{8}$	$8\frac{3}{4}$	13.4	13.4
$1\frac{1}{8}$	$4\frac{3}{4}$	2.3	2.3	$2\frac{1}{4}$	$9\frac{1}{8}$	15.8	15.8
$1\frac{1}{4}$	$5\frac{1}{4}$	3.0	3.0	$2\frac{3}{8}$	$9\frac{3}{8}$	17.5	17.5
$1\frac{3}{8}$	$5\frac{3}{4}$	4.2	4.2	$2\frac{1}{2}$	$9\frac{3}{8}$	20.0	20.0
$1\frac{1}{2}$	$6\frac{1}{4}$	5.2	5.2				

WEIGHT OF

LARGER SIZES OF FORGED HEXAGON NUTS

Diameter of bolt, in inches.	Weight, in lbs.	Diameter of bolt, in inches.	Weight, in lbs.
2¼	8	3⅛	20
2⅜	9	3¼	22
2½	9½	3⅜	23
2⅝	11	3½	24
2¾	13½	3⅝	25
2⅞	14	3¾	27
3	17	4	29

NOTE.—The above is the weight of iron required to forge one nut of the sizes given.

Weight, in lbs., of

NUT AND BOLT HEADS.

For common-sized nuts and heads, the following table is close enough for estimating the weights.

HEAD AND NUT.			HEAD AND NUT.		
Diameter of bolt.	Square.	Hexagon.	Diameter of bolt.	Square.	Hexagon.
¼	0.021	0.017	1¼	2.56	2.14
⅜	0.70	0.57	1½	4.42	3.77
½	0.164	0.128	1¾	7.00	5.62
⅝	0.321	0.267	2	10.5	8.75
¾	0.55	0.43	2½	21.0	17.2
⅞	0.88	0.73	3	36.4	28.8
1	1.31	1.1			

Weight of

ONE SQ. FOOT OF SHEET IRON OR STEEL.

Birmingham Gauge.

No. of gauge.	Thickness, in inches. In decimals.	Thickness, in inches. In fractions.	Iron.	Steel.
0000	0.454	$\frac{29}{64}$	18.35	18.54
000	0.425	$\frac{55}{128}$	17.18	17.35
00	0.380	$\frac{49}{128}$	15.36	15.51
0	0.340	$\frac{11}{32}$	13.74	13.87
1	0.300	$\frac{19}{64}$	12.13	12.25
2	0.284	$\frac{9}{32}$	11.48	11.59
3	0.259	$\frac{33}{128}$	10.47	10.57
4	0.238	$\frac{31}{128}$	9.62	9.72
5	0.220	$\frac{7}{32}$	8.89	8.98
6	0.203		8.21	8.29
7	0.180	$\frac{23}{128}$	7.27	7.35
8	0.165		6.70	6.74
9	0.148	$\frac{19}{128}$	5.98	6.04
10	0.134		5.42	5.47
11	0.120	$\frac{15}{128}$	4.85	4.90
12	0.109	$\frac{7}{64}$	4.41	4.45
13	0.095	$\frac{3}{32}$	3.84	3.88
14	0.083		3.35	3.39
15	0.072		2.91	2.94
16	0.065	$\frac{1}{16}$	2.63	2.65
17	0.058		2.34	2.37
18	0.049		1.98	2.00
19	0.042		1.70	1.71
20	0.035		1.41	1.43
21	0.032	$\frac{1}{32}$	1.29	1.30
22	0.028		1.13	1.14
23	0.025		1.01	1.02
24	0.022		0.889	0.898
25	0.020		0.808	0.816
26	0.018		0.722	0.735
27	0.016	$\frac{1}{64}$	0.647	0.653
28	0.014		0.568	0.572
29	0.013		0.525	0.531
30	0.012		0.485	0.490
31	0.010		0.404	0.408
32	0.009		0.364	0.367
33	0.008	$\frac{1}{128}$	0.323	0.326
34	0.007		0.283	0.286
35	0.005		0.202	0.204

Weight of

ONE SQ. FOOT OF SHEET IRON OR STEEL.

American Gauge.

No. of gauge.	Thickness, in inches.		Iron.	Steel.
	In decimals.	In fractions.		
0000	0.46	$\frac{15}{32}$	18.63	18.87
000	0.41	$\frac{13}{32}$	16.58	16.80
00	0.365	$\frac{23}{64}$	14.77	15.00
0	0.325	$\frac{21}{64}$	13.15	13.32
1	0.289	$\frac{19}{64}$	11.70	11.86
2	0.257	$\frac{17}{64}$	10.43	10.57
3	0.229	$\frac{15}{64}$	9.29	9.42
4	0.204	$\frac{13}{64}$	8.27	8.38
5	0.182	$\frac{3}{16}$	7.37	7.46
6	0.162	$\frac{11}{64}$	6.56	6.64
7	0.144	$\frac{9}{64}$	5.84	5.92
8	0.128	$\frac{1}{8}$	5.20	5.27
9	0.114		4.63	4.69
10	0.102	$\frac{7}{64}$	4.12	4.18
11	0.091	$\frac{3}{32}$	3.67	3.72
12	0.080		3.27	3.31
13	0.072	$\frac{5}{64}$	2.92	2.95
14	0.064	$\frac{1}{16}$	2.59	2.63
15	0.057		2.31	2.34
16	0.050		2.05	2.08
17	0.045	$\frac{3}{64}$	1.83	1.86
18	0.040		1.63	1.65
19	0.036		1.45	1.47
20	0.032	$\frac{1}{32}$	1.29	1.31
21	0.028		1.15	1.16
22	0.025		1.03	1.04
23	0.023		0.91	0.92
24	0.020		0.81	0.82
25	0.018		0.72	0.73
26	0.016	$\frac{1}{64}$	0.64	0.65
27	0.014		0.57	0.58
28	0.013		0.51	0.52
29	0.011		0.46	0.47
30	0.010		0.41	0.41
31	0.009		0.36	0.37
32	0.008	$\frac{1}{128}$	0.32	0.33
33	0.007		0.29	0.29
34	0.006		0.25	0.26
35	0.005		0.23	0.23

AMERICAN AND BIRMINGHAM WIRE GAUGES

Thickness, in inches.

HASWELL

No. of gauge.	Thickness of American gauge.	Thickness of Birmingham gauge.	No. of gauge.	Thickness of American gauge.	Thickness of Birmingham gauge.
0000	0.46	0.454	17	0.0452	0.058
000	0.4096	0.425	18	0.0403	0.049
00	0.3648	0.38	19	0.0359	0.042
0	0.3248	0.34	20	0.0319	0.035
1	0.2893	0.30	21	0.0284	0.032
2	0.2576	0.284	22	0.0253	0.028
3	0.2294	0.259	23	0.0225	0.025
4	0.2043	0.238	24	0.0201	0.022
5	0.1819	0.22	25	0.0179	0.02
6	0.1620	0.203	26	0.0160	0.018
7	0.1443	0.18	27	0.0142	0.016
8	0.1285	0.165	28	0.0126	0.014
9	0.1144	0.148	29	0.0112	0.013
10	0.1019	0.134	30	0.01	0.012
11	0.0907	0.12	31	0.0089	0.01
12	0.0808	0.109	32	0.0079	0.009
13	0.0719	0.095	33	0.007	0.008
14	0.0641	0.083	34	0.0063	0.007
15	0.057	0.072	35	0.0056	0.005
16	0.0508	0.065	36	0.005	0.004

CAST-IRON PIPE.

Weight of a lineal foot.

Bore, in inches.	Thickness of metal, in inches.								
	1/4	3/8	1/2	5/8	3/4	7/8	1	1 1/8	1 1/4
	LBS.	LBS.	LBS.	LBS.	LBS.	LBS.	LBS.	LBS.	LBS.
2	5.5	8.7	12.3	16.1	20.3	24.7	29.5	34.5	39.9
2½	6.8	10.6	14.7	19.2	24.0	29.0	34.4	40.0	46.0
3	7.9	12.4	17.2	22.2	27.6	32.3	39.3	45.6	52.2
3½	9.2	14.3	19.6	25.3	31.3	37.6	44.2	51.0	58.3
4	10.4	16.1	22.1	28.4	35.0	41.9	49.1	56.6	64.4
4½	11.7	18.0	24.5	31.5	38.7	46.2	54.0	62.1	70.6
5	12.9	19.8	27.0	34.5	42.3	50.5	59.9	67.7	76.7
5½	14.1	21.6	29.5	37.6	46.0	54.8	63.8	73.2	82.9
6	15.3	23.5	31.9	40.7	49.7	59.1	68.7	78.7	89.0
7	17.8	27.2	36.9	46.8	57.1	67.7	78.5	89.8	101.0
8	20.3	30.8	41.7	52.9	64.4	76.2	88.4	101.0	114.0
9	22.7	34.5	46.6	59.1	71.8	84.8	98.2	112.0	126.0
10	25.2	38.2	51.5	65.2	79.2	93.4	108.0	123.0	138.0
11	27.6	41.9	56.5	71.3	86.5	102.0	118.0	134.0	150.0
12	30.1	45.6	61.4	77.5	93.9	111.0	128.0	145.0	163.0
13	32.5	49.2	66.3	83.6	101.0	119.0	138.0	156.0	175.0
14	35.0	52.9	71.2	89.7	109.0	128.0	147.0	167.0	187.0
15	37.4	56.6	76.1	95.9	116.0	136.0	157.0	178.0	199.0
16	39.1	60.3	81.0	102.0	123.0	145.0	167.0	189.0	212.0
18	44.8	67.7	90.9	114.0	138.0	162.0	187.0	211.0	236.0
20	49.7	75.2	101.0	127.0	153.0	179.0	206.0	233.0	261.0
22	54.6	82.6	111.0	139.0	168.0	197.0	226.0	255.0	285.0
24	59.6	89.9	120.0	151.0	182.0	214.0	245.0	278.0	310.0
26	64.5	97.3	131.0	164.0	198.0	231.0	266.0	300.0	335.0
28	69.4	105.0	140.0	176.0	212.0	249.0	286.0	323.0	360.0
30	74.2	112.0	150.0	188.0	227.0	266.0	305.0	345.0	384.0

NOTE.—For each joint, add a foot to length of pipe.

WROUGHT-IRON WELDED TUBES.

For steam, gas, or water.

1¼ inch and below, butt welded; proved to 300 pounds per square inch, hydraulic pressure.

1½ inch and above, lap welded; proved to 500 pounds per square inch, hydraulic pressure.

TABLE OF STANDARD DIMENSIONS.

MORRIS, TASKER & CO., LIMITED.

Inside diameter, in inches.	Actual outside diameter, in inches.	Thickness, in inches.	Actual inside diameter, in inches.	Internal area, in inches.	External area, in inches.	Weight per foot of length, in pounds.	Number of threads per inch of screw.
⅛	0.405	0.068	0.270	0.0572	0.129	0.243	27
¼	0.54	0.088	0.361	0.1041	0.229	0.422	18
⅜	0.675	0.091	0.494	0.1916	0.358	0.561	18
½	0.84	0.109	0.623	0.3048	0.554	0.845	14
¾	1.05	0.113	0.824	0.5333	0.866	1.126	14
1	1.315	0.134	1.048	0.8627	1.357	1.670	11½
1¼	1.66	0.140	1.380	1.496	2.164	2.258	11½
1½	1.9	0.145	1.611	2.038	2.835	2.694	11½
2	2.375	0.154	2.067	3.355	4.430	3.667	11½
2½	2.875	0.204	2.468	4.783	6.491	5.773	8
3	3.5	0.217	3.067	7.388	9.621	7.547	8
3½	4.0	0.226	3.548	9.887	12.566	9.055	8
4	4.5	0.237	4.026	12.730	15.904	10.728	8
4½	5.0	0.247	4.508	15.939	19.635	12.492	8
5	5.563	0.259	5.045	19.990	24.299	14.564	8
6	6.625	0.280	6.065	28.889	34.471	18.767	8
7	7.625	0.301	7.023	38.737	45.663	23.410	8
8	8.625	0.322	7.982	50.039	58.426	28.348	8
9	9.688	0.344	9.001	63.633	73.715	34.077	8
10	10.75	0.366	10.019	78.838	90.762	40.641	8

WROUGHT-IRON WELDED TUBES.

Extra strong.

Nominal Diameter.	Actual outside diameter.	Thickness, extra strong.	Thickness, double extra strong.	Actual inside diameter, extra strong.	Actual inside diameter, double extra strong.
$\frac{1}{8}$	0.405	0.100		0.205	
$\frac{1}{4}$	0.54	0.123		0.294	
$\frac{3}{8}$	0.675	0.127		0.421	
$\frac{1}{2}$	0.84	0.149	0.298	0.542	0.244
$\frac{3}{4}$	1.05	0.157	0.314	0.736	0.422
1	1.315	0.182	0.364	0.951	0.587
$1\frac{1}{4}$	1.66	0.194	0.388	1.272	0.884
$1\frac{1}{2}$	1.9	0.203	0.406	1.494	1.088
2	2.375	0.221	0.442	1.933	1.491
$2\frac{1}{2}$	2.875	0.280	0.560	2.315	1.755
3	3.5	0.304	0.608	2.892	2.284
$3\frac{1}{2}$	4.0	0.321	0.642	3.358	2.716
4	4.5	0.341	0.682	3.818	3.136

SHIP SPIKES.

Number in one hundred pounds.

Size, in inches.	Length, in inches.	Number in 100 lbs.	Size, in inches.	Length, in inches.	Number in 100 lbs.	Size, in inches.	Length, in inches.	Number in 100 lbs.
$\frac{1}{4}$	3	1910	$\frac{7}{16}$	5	461	$\frac{9}{16}$	7	190
$\frac{1}{4}$	$3\frac{1}{2}$	1585	$\frac{7}{16}$	$5\frac{1}{2}$	423	$\frac{9}{16}$	$7\frac{1}{2}$	180
$\frac{1}{4}$	4	1326	$\frac{7}{16}$	6	402	$\frac{9}{16}$	8	170
$\frac{1}{4}$	$4\frac{1}{2}$	1223	$\frac{7}{16}$	$6\frac{1}{2}$	321	$\frac{9}{16}$	$8\frac{1}{2}$	160
$\frac{1}{4}$	5	1025	$\frac{1}{2}$	5	340	$\frac{9}{16}$	9	150
$\frac{5}{16}$	3	1010	$\frac{1}{2}$	$5\frac{1}{2}$	312	$\frac{9}{16}$	10	140
$\frac{5}{16}$	$3\frac{1}{2}$	963	$\frac{1}{2}$	6	298	$\frac{5}{8}$	8	140
$\frac{5}{16}$	4	810	$\frac{1}{2}$	$6\frac{1}{2}$	280	$\frac{5}{8}$	9	120
$\frac{5}{16}$	$4\frac{1}{2}$	605	$\frac{1}{2}$	7	261	$\frac{5}{8}$	10	110
$\frac{5}{16}$	5	583	$\frac{1}{2}$	$7\frac{1}{2}$	240	$\frac{5}{8}$	11	100
$\frac{5}{16}$	6	521	$\frac{1}{2}$	8	223	$\frac{3}{4}$	10	80
$\frac{7}{16}$	4	542	$\frac{9}{16}$	6	221	$\frac{3}{4}$	15	60
$\frac{7}{16}$	$4\frac{1}{2}$	503	$\frac{9}{16}$	$6\frac{1}{2}$	200			

NUMBER OF NAILS AND TACKS PER POUND.

NAILS.			TACKS.		
Title.	Length, in inches.	No. nails per lb.	Title.	Length, in inches.	No. tacks per lb.
3 penny fine.	1⅛	760	1 oz.	⅛	16,000
3 "	1¼	480	1½ "	3/16	10,666
4 "	1½	300	2 "	¼	8,000
5 "	1¾	200	2½ "	5/16	6,400
6 "	2	160	3 "	⅜	5,333
7 "	2¼	128	4 "	7/16	4,000
8 "	2½	92	6 "	9/16	2,666
9 "	2¾	72	8 "	⅝	2,000
10 "	3	60	10 "	11/16	1,600
12 "	3¼	44	12 "	¾	1,333
16 "	3½	32	14 "	13/16	1,143
20 "	4	24	16 "	⅞	1,000
30 "	4½	18	18 "	15/16	888
40 "	5	14	20 "	1	800
50 "	5½	12	22 "	1 1/16	727
60 "	6	10	24 "	1⅛	666
6 " fence.	2	80			
8 "	2½	50			
10 "	3	34			
12 "	3¼	29			

5 pounds of 4 penny, or 3¾ pounds of 3 penny, will lay 1000 shingles; 5¾ pounds of 3 penny fine will put on 1000 laths, 4 nails to the lath.

RAILROAD SPIKES.

Length, in inches.	Thickness, in inches.	No. in 100 lbs.	Length, in inches.	Thickness, in inches.	No. in 100 lbs.
4½	7/16	351	5½	½	237
4½	½	267	5½	9/16	193
5	⅜	473	5½	⅝	146
5	7/16	326	6	½	207
5	½	260	6	9/16	175
5	9/16	197	6	⅝	131
5	⅝	172			

RAILROAD BARS.

Table showing the number of tons per mile corresponding to the following weight of rails per lineal yard. Ton of 2240 pounds.

Weight per yard, in lbs.	Tons per mile.	Weight per yard, in lbs.	Tons per mile.
8	$12.\frac{1280}{2240}$	52	$81.\frac{1600}{2240}$
12	$18.\frac{1920}{2240}$	56	88
16	$25.\frac{320}{2240}$	57	$89.\frac{1280}{2240}$
25	$39.\frac{640}{2240}$	60	$94.\frac{640}{2240}$
30	$47.\frac{320}{2240}$	62	$97.\frac{960}{2240}$
35	55	64	$100.\frac{1280}{2240}$
40	$62.\frac{1920}{2240}$	65	$102.\frac{320}{2240}$
45	$70.\frac{1600}{2240}$	68	$106.\frac{1920}{2240}$
50	$78.\frac{1230}{2240}$	70	110

Calculated for "single track" (2 rails).

Multiply the pounds per yard by $1\frac{4}{7}$, and the result will be the number of tons (of 2240 pounds) per mile of single track.

RAILROAD SPLICE OR "FISH" JOINTS.

The ordinary length of splice plates is 23″ or 24″, with 4 bolts of $\frac{3}{4}$″ diameter to each pair of plates. The average weight of the plates is 16 pounds, and of the 4 bolts (with *single* nuts), 4 pounds, making 20 pounds total weight per "joint." If double or "jam" nuts are used, the weight of the 4 bolts will be $5\frac{1}{2}$ pounds, or $21\frac{1}{2}$ pounds per joint.

"SINGLE TRACK."

Lengths of rail, in feet.	Number of joints per mile.	Pounds of plates per mile.	Pounds of bolts per mile.	Total weight per mile.
18	588	9408	2352	11,760
21	528	8448	2112	10,560
24	440	7040	1760	8,800
25	423	6768	1692	8,460
27	391	6256	1564	7,820
30	352	5632	1408	7,040

NOTE.—If double nuts are used, add $37\frac{1}{2}$ per cent. to the weight of the bolts.

NOTE ON BRICK ARCHES FOR FLOORS.

The approximate number of bricks, and the cost of brick work in arches for floors, will depend somewhat upon the size and cost per thousand of bricks.

With bricks $8\frac{1}{4} \times 4 \times 2$, and joints of mortar from $\frac{1}{8}''$ to $\frac{1}{4}''$ between them, edgewise arches will require about 8 bricks per square foot of floor, and endwise arches will require $16\frac{1}{2}$.

Estimating the average cost of hard brick at $10 per thousand, and the cost of laying, including mortar, centres, scaffolding, etc., at $10 per thousand more, or $20 per thousand in place, the edgewise arches will cost 16 cents per square foot, and the endwise arches 33 cents per square foot, put up complete.

WEIGHTS OF MATERIALS.

	Per cubic foot.
Water	62.3
Fire brick	137.0
Brick work	112.0
Coal, anthracite, solid	100.0
Coal, anthracite, broken	57.0
Coal, bituminous	77.0– 90.0
Coke	62.0–104.0
Granite	164.0–172.0
Plaster of Paris	73.5
Limestone	169.0–175.0
Masonry	116.0–144.0
Sandstone	144.0
Slate	178.0
Common gravel	109.0
Mud	102.0
Mortar	98.0
Concrete	125.0
Common soil	137.0
Glass	165.0

1 bushel of bituminous coal weighs 80 pounds.
28 bushels = 1 ton of 2240 pounds.

WEIGHT OF TIMBER.

	Lbs. per cubic foot.	Lbs. per foot, B. M.	Relative strength for cross breaking.	Crushing weight per sq. inch in tons of 2000 lbs.
Ash	47	3.9	149	4.3
Beech, white			115	
Beech, red	43	3.6	144	4.6
Chestnut	33	2.8	112	
Cedar, American white	50	4.2	63	2.8
Elm	34	2.8		5.1
Hemlock			95	
Locust	44	3.7		
Maple	49	4.1		
White oak	45	3.8	145	2.8
Live oak	70	5.8	155	
White pine	30	2.5	102	2.5
Yellow pine	33	2.8	98	2.7
Spruce			86	
Black walnut	42	3.5	121	3.0

PLASTERING.

The plastering of inside walls of buildings generally consists of three separate coats of mortar.

A plasterer, aided by one or two laborers, can average from 100 to 150 square yards a day of first coat; 90 to 100 yards of second coat; and about 50 yards for the third coat.

One thousand laths, $1\frac{1}{2}'' \times 4'$, will cover 660 square feet, and a carpenter can nail up laths at the rate of 50 square yards per day, in common square rooms.

AMERICAN SLATING.

Slating is estimated by the "square," which is the quantity required to cover 100 square feet. The slates are usually laid so that the third laps the first three inches. Therefore to compute the number of slates of a given size required per square: Subtract 3″ from the length of the slate, multiply the remainder by the width, and divide by 2. This will give the number of square inches covered per slate; divide 14,400 (the number of square inches in a square) by the number so found, and the result will be the number of slates required.

The following table gives the number of slates per square for the usual sizes:

NUMBER OF SLATES PER SQUARE.

Size, in inches.	Pieces per square.	Size, in inches.	Pieces per square.	Size, in inches.	Pieces per square.
6 × 12	533	8 × 16	277	12 × 20	141
7 × 12	457	9 × 16	246	14 × 20	121
8 × 12	400	10 × 16	221	11 × 22	137
9 × 12	355	9 × 18	213	12 × 22	126
7 × 14	374	10 × 18	192	14 × 22	108
8 × 14	327	12 × 18	160	12 × 24	114
9 × 14	291	10 × 20	169	14 × 24	98
10 × 14	261	11 × 20	154	16 × 24	86

The weight of slate per cubic foot is about 174 pounds, or per square foot of various thicknesses as follows:

Thickness, in inches.	Weight, in lbs.	Thickness, in inches.	Weight, in lbs.	Thickness, in inches.	Weight, in lbs.
$\frac{1}{8}$	1.81	$\frac{1}{4}$	3.62	$\frac{1}{2}$	7.25
$\frac{3}{16}$	2.71	$\frac{3}{8}$	5.43		

The weight of slating laid per square foot of surface covered will, of course, depend on the size used. The weight of 10 × 18 slate, $\frac{3}{16}$ thick, for example, per square foot of roof, would be 5.86 pounds.

SHINGLING.

An average shingle $7\frac{1}{2}$″ wide in $8\frac{1}{2}$″ courses shows 64 □″, making 3 shingles to a square foot of roof, including waste. They are usually laid in 3 thicknesses.

PAINTING AND GLAZING.

Painting is measured by the superficial yard, girting every part of the work that is covered by paint, and allowing an addition to the actual surface for covering deep quirks of moulding. Generally estimates are made for each coat of paint at a certain price per superficial yard.

WINDOW GLASS.

NUMBER OF LIGHTS PER BOX OF FIFTY FEET.

Inches.	No.	Inches.	No.	Inches.	No.	Inches.	No.
6 × 8	150	12 × 18	33	16 × 44	10	26 × 32	9
7 × 9	115	12 × 20	30	18 × 20	20	26 × 34	8
8 × 10	90	12 × 22	27	18 × 22	18	26 × 36	8
8 × 11	82	12 × 24	25	18 × 24	17	26 × 40	7
8 × 12	75	12 × 26	23	18 × 26	15	26 × 42	7
8 × 13	70	12 × 28	21	18 × 28	14	26 × 44	6
8 × 14	64	12 × 30	20	18 × 30	13	26 × 48	6
8 × 15	60	12 × 32	18	18 × 32	13	26 × 50	6
8 × 16	55	12 × 34	17	18 × 34	12	26 × 54	5
9 × 11	72	13 × 14	40	18 × 36	11	26 × 58	5
9 × 12	67	13 × 16	35	18 × 38	11	28 × 30	9
9 × 13	62	13 × 18	31	18 × 40	10	28 × 32	8
9 × 14	57	13 × 20	28	18 × 44	9	28 × 34	8
9 × 15	53	13 × 22	25	20 × 22	16	28 × 36	7
9 × 16	50	13 × 24	23	20 × 24	15	28 × 38	7
9 × 17	47	13 × 26	21	20 × 26	14	28 × 40	6
9 × 18	44	13 × 28	19	20 × 28	13	28 × 44	6
9 × 20	40	13 × 30	18	20 × 30	12	28 × 46	6
10 × 12	60	14 × 16	32	20 × 32	11	28 × 50	5
10 × 13	55	14 × 18	29	20 × 34	11	28 × 52	5
10 × 14	52	14 × 20	26	20 × 36	10	28 × 56	4
10 × 15	48	14 × 22	23	20 × 38	9	30 × 36	7
10 × 16	45	14 × 24	22	20 × 40	9	30 × 40	6
10 × 17	42	14 × 26	20	20 × 44	8	30 × 42	6
10 × 18	40	14 × 28	18	20 × 46	8	30 × 44	5
10 × 20	36	14 × 30	17	20 × 48	8	30 × 46	5
10 × 22	33	14 × 32	16	20 × 50	7	30 × 48	5
10 × 24	30	14 × 34	15	20 × 60	6	30 × 50	5
10 × 26	28	14 × 36	14	22 × 24	14	30 × 54	4
10 × 28	26	14 × 40	13	22 × 26	13	30 × 56	4
10 × 30	24	14 × 44	11	22 × 28	12	30 × 60	4
10 × 32	22	15 × 18	27	22 × 30	11	32 × 42	5
10 × 34	21	15 × 20	24	22 × 32	10	32 × 44	5
11 × 13	50	15 × 22	22	22 × 34	10	32 × 46	5
11 × 14	47	15 × 24	20	22 × 36	9	32 × 48	5
11 × 15	44	15 × 26	18	22 × 38	9	32 × 50	4
11 × 16	41	15 × 28	17	22 × 40	8	32 × 54	4
11 × 17	39	15 × 30	16	22 × 44	8	32 × 56	4
11 × 18	36	15 × 32	15	22 × 46	7	32 × 60	4
11 × 20	33	16 × 18	25	22 × 50	7	34 × 40	5
11 × 22	30	16 × 20	23	24 × 28	11	34 × 44	5
11 × 24	27	16 × 22	20	24 × 30	10	34 × 46	5
11 × 26	25	16 × 24	19	24 × 32	9	34 × 50	4
11 × 28	23	16 × 26	17	24 × 36	8	34 × 52	4
11 × 30	21	16 × 28	16	24 × 40	8	34 × 56	4
11 × 32	20	16 × 30	15	24 × 44	7	36 × 44	5
11 × 34	19	16 × 32	14	24 × 46	7	36 × 50	4
12 × 14	43	16 × 34	13	24 × 48	6	36 × 56	4
12 × 15	40	16 × 36	12	24 × 50	6	36 × 60	3
12 × 16	38	16 × 38	12	24 × 54	5	36 × 64	3
12 × 17	35	16 × 40	11	24 × 56	5	40 × 60	3

SKYLIGHT AND FLOOR GLASS.

LENNOX PLATE GLASS CO. WARD & CO., AGENTS, PHILADELPHIA.

Weight per cubic foot, 156 pounds.

WEIGHT PER SQUARE FOOT.

Thickness, in inches.	Weight, in lbs.	Thickness, in inches.	Weight, in lbs.	Thickness, in inches.	Weight, in lbs.
$\frac{1}{8}$	1.62	$\frac{3}{8}$	4.88	$\frac{3}{4}$	9.75
$\frac{3}{16}$	2.43	$\frac{1}{2}$	6.50	1	13.00
$\frac{1}{4}$	3.25	$\frac{5}{8}$	8.13		

FLAGGING.

Weight per cubic foot, 168 pounds.

WEIGHT PER SQUARE FOOT.

Thickness, in inches.	Weight, in lbs.	Thickness, in inches.	Weight, in lbs.	Thickness, in inches.	Weight, in lbs.
1	14	4	56	7	98
2	28	5	70	8	112
3	42	6	84		

BRICK WORK AND MASONRY.

Stone work is estimated by the perch of 25 cubic feet. Brick work is estimated by the thousand, and for various thicknesses of wall runs as follows:

9″ wall, or 1 brick in thickness, 14 bricks per superficial foot.
13″ wall, or $1\frac{1}{2}$ bricks in thickness, 21 bricks per superficial foot.
18″ wall, or 2 bricks in thickness, 28 bricks per superficial foot.
22″ wall, or $2\frac{1}{2}$ bricks in thickness, 35 bricks per superficial foot.

For each additional half brick in thickness count seven (7) bricks per superficial foot.

One square yard of paving requires 36 bricks when laid flat, or 82 when laid on edge.

A 9″ wall will weigh 84 pounds per square foot of side surface; a 13″ wall, 121 pounds; an 18″ wall, 168 pounds; assuming weight per cubic foot of brick work at 112 pounds.

GALVANIZED AND BLACK IRON.

Weight, in pounds, per square foot of galvanized sheet iron, both flat and corrugated.

The numbers and thicknesses are those of the iron before it is galvanized. When a flat sheet (the ordinary size of which is from 2 to $2\frac{1}{2}$ feet in width by 6 to 8 feet in length) is converted into a corrugated one, with corrugations 5 inches wide from centre to centre, and about an inch deep (the common sizes), its width is thereby reduced about $\frac{1}{10}$ part, or from 30 to 27 inches; and consequently the weight per square foot of area covered is increased about $\frac{1}{9}$ part. When the corrugated sheets are laid upon a roof, the overlapping of about $2\frac{1}{2}$ inches along their sides, and of 4 inches along their ends, diminishes the covered area about $\frac{1}{7}$ part more; making their weight per square foot of roof about $\frac{1}{6}$ part greater than before. Or the weight of corrugated iron per square foot in place on a roof is about $\frac{1}{3}$ greater than that of the flat sheets of above sizes of which it is made.

No. by Birmingham wire gauge.	BLACK.		GALVANIZED.		
	Thickness, in inches.	Flat, in lbs.	Flat, in lbs.	Corrugated, in lbs.	Corrugated, on roof, in lbs.
30	0.012	0.485	0.806	0.896	1.08
29	0.013	0.526	0.857	0.952	1.14
28	0.014	0.565	0.897	0.997	1.20
27	0.016	0.646	0.978	1.09	1.30
26	0.018	0.722	1.06	1.18	1.41
25	0.020	0.808	1.14	1.27	1.52
24	0.022	0.889	1.22	1.36	1.62
23	0.025	1.01	1.34	1.49	1.79
22	0.028	1.13	1.46	1.62	1.95
21	0.032	1.29	1.63	1.81	2.17
20	0.035	1.41	1.75	1.94	2.33
19	0.042	1.69	2.03	2.26	2.71
18	0.049	1.98	2.32	2.58	3.09
17	0.058	2.34	2.68	2.98	3.57
16	0.065	2.63	2.96	3.29	3.95
15	0.072	2.91	3.25	3.61	4.33
14	0.083	3.36	3.69	4.10	4.92
13	0.095	3.84	4.18	4.64	5.57

NOTE.—The galvanizing of sheet iron adds about one-third of a pound to its weight per square foot.

Nos. 20 to 22 are the usual sizes for roof coverings.

TABLES OF MEASURES

COMPILED FROM VARIOUS SOURCES.

TABLE OF

DECIMAL PARTS OF A FOOT FOR EACH ONE-THIRTY-SECOND OF AN INCH.

Inch.	0	1	2	3	4	5	6	7	8	9	10	11
0	0000	0833	1667	2500	3333	4167	5000	5833	6667	7500	8333	9167
$\frac{1}{32}$	0026	0859	1693	2526	3359	4193	5026	5859	6693	7526	8359	9193
$\frac{1}{16}$	0052	0885	1719	2552	3385	4219	5052	5885	6719	7552	8385	9219
$\frac{3}{32}$	0078	0911	1745	2578	3411	4245	5078	5911	6745	7578	8411	9245
$\frac{1}{8}$	0104	0938	1772	2604	3438	4271	5104	5938	6771	7604	8438	9271
$\frac{5}{32}$	0130	0964	1797	2630	3464	4297	5130	5964	6797	7630	8464	9297
$\frac{3}{16}$	0156	0990	1823	2656	3490	4323	5156	5990	6823	7656	8490	9323
$\frac{7}{32}$	0182	1016	1849	2682	3516	4349	5182	6016	6849	7682	8516	9349
$\frac{1}{4}$	0208	1042	1875	2708	3542	4375	5208	6042	6875	7708	8542	9375
$\frac{9}{32}$	0234	1068	1901	2734	3568	4401	5234	6068	6901	7734	8568	9401
$\frac{5}{16}$	0260	1094	1927	2760	3594	4427	5260	6094	6927	7760	8594	9427
$\frac{11}{32}$	0286	1120	1953	2786	3620	4453	5286	6120	6953	7786	8620	9453
$\frac{3}{8}$	0313	1146	1979	2813	3646	4479	5313	6146	6979	7813	8646	9479
$\frac{13}{32}$	0339	1172	2005	2839	3672	4505	5339	6172	7005	7839	8672	9505
$\frac{7}{16}$	0365	1198	2031	2865	3698	4531	5365	6198	7031	7865	8698	9531
$\frac{15}{32}$	0391	1224	2057	2891	3724	4557	5391	6224	7057	7891	8724	9557
Inch.	0	1	2	3	4	5	6	7	8	9	10	11

TABLE OF

DECIMAL PARTS OF A FOOT FOR EACH ONE-THIRTY-SECOND OF AN INCH.

Inch.	0	1	2	3	4	5	6	7	8	9	10	11
$\frac{1}{2}$	0417	1250	2083	2917	3750	4583	5417	6250	7083	7917	8750	9583
$\frac{17}{32}$	0443	1276	2109	2943	3776	4609	5443	6276	7109	7943	8776	9609
$\frac{9}{16}$	0469	1302	2135	2969	3802	4635	5469	6302	7135	7969	8802	9635
$\frac{19}{32}$	0495	1328	2161	2995	3828	4661	5495	6328	7161	7995	8828	9661
$\frac{5}{8}$	0521	1354	2188	3021	3854	4688	5521	6354	7188	8021	8854	9688
$\frac{21}{32}$	0547	1380	2214	3047	3880	4714	5547	6380	7214	8047	8880	9714
$\frac{11}{16}$	0573	1406	2240	3073	3906	4740	5573	6406	7240	8073	8906	9740
$\frac{23}{32}$	0599	1432	2266	3099	3932	4766	5599	6432	7266	8099	8932	9766
$\frac{3}{4}$	0625	1458	2292	3125	3958	4792	5625	6458	7292	8125	8958	9792
$\frac{25}{32}$	0651	1484	2318	3151	3984	4818	5651	6484	7318	8151	8984	9818
$\frac{13}{16}$	0677	1510	2344	3177	4010	4844	5677	6510	7344	8177	9010	9844
$\frac{27}{32}$	0703	1536	2370	3203	4036	4870	5703	6536	7370	8203	9036	9870
$\frac{7}{8}$	0729	1563	2396	3229	4063	4896	5729	6563	7396	8229	9063	9896
$\frac{29}{32}$	0755	1589	2422	3255	4089	4922	5755	6589	7422	8255	9089	9922
$\frac{15}{16}$	0781	1615	2448	3281	4115	4948	5781	6615	7448	8281	9115	9948
$\frac{31}{32}$	0807	1641	2474	3307	4141	4974	5807	6641	7474	8307	9141	9974
Inch.	0	1	2	3	4	5	6	7	8	9	10	11

TABLES OF

DECIMAL PARTS OF AN INCH FOR EACH ONE-SIXTY-FOURTH.

$\frac{1}{64}$	0.015625	$\frac{19}{64}$	0.2969	$\frac{5}{8}$	0.6250
$\frac{1}{32}$	0.03125	$\frac{5}{16}$	0.3125	$\frac{21}{32}$	0.6562
$\frac{3}{64}$	0.04687	$\frac{11}{32}$	0.3438	$\frac{43}{64}$	0.6719
$\frac{1}{16}$	0.0625	$\frac{23}{64}$	0.3594	$\frac{11}{16}$	0.6875
$\frac{5}{64}$	0.07812	$\frac{3}{8}$	0.3750	$\frac{23}{32}$	0.7188
$\frac{3}{32}$	0.09375	$\frac{13}{32}$	0.4063	$\frac{3}{4}$	0.7500
$\frac{7}{64}$	0.10937	$\frac{27}{64}$	0.4219	$\frac{25}{32}$	0.7812
$\frac{1}{8}$	0.1250	$\frac{7}{16}$	0.4375	$\frac{13}{16}$	0.8125
$\frac{9}{64}$	0.1406	$\frac{15}{32}$	0.4688	$\frac{27}{32}$	0.8437
$\frac{5}{32}$	0.1563	$\frac{31}{64}$	0.4844	$\frac{7}{8}$	0.8750
$\frac{11}{64}$	0.1718	$\frac{1}{2}$	0.5000	$\frac{57}{64}$	0.8906
$\frac{3}{16}$	0.1875	$\frac{17}{32}$	0.5312	$\frac{29}{32}$	0.9062
$\frac{7}{32}$	0.2187	$\frac{35}{64}$	0.5469	$\frac{15}{16}$	0.9375
$\frac{15}{64}$	0.2344	$\frac{9}{16}$	0.5625	$\frac{61}{64}$	0.9531
$\frac{1}{4}$	0.2500	$\frac{19}{32}$	0.5938	$\frac{31}{32}$	0.9688
$\frac{9}{32}$	0.2813	$\frac{39}{64}$	0.6094	$\frac{63}{64}$	0.9844

MEASUREMENTS OF LENGTH.

Miles.	Rods.	Yards.	Feet.	Inches.
1.	320.	1760.	5280.	63360.
0.003125	1.	5.5	16.5	198.
0.000568	0.1818	1.	3.	36.
0.00019	0.0606	0.0333	1.	12.
0.0000157	0.00505	0.0277	0.08333	1.

Prussian foot = 12.356 inches. Prussian mile = 4.6804 English miles. German mile = 4.6105 English miles. Russian verst = 3500 feet = 0.6629 English mile.

MEASUREMENT OF WEIGHTS.

Tons.	Cwts.	Pounds.	Ounces.
1.	20.	2240.	35840.
0.050	1.	112.	1792.
	0.0089	1.	16.
		0.0625	1.

1 pound = 27.7 cubic inches of distilled water at 40° Fahrenheit.

MEASUREMENT OF CAPACITY.

Cubic yards.	Barrels.	Bushels.	Cubic feet.	Gallons.	Cubic inches.
1.	5.6103	25.2467	27.	201.97	46656.
0.1782	1.	4.5	4.8125	36.	8316.
0.0396	0.222	1.	1.2438	8.	2150.
	0.2078	0.804	1.	7.476	1728.
	0.0277	0.125	0.13369	1.	231.
			0.000578	0.00433	1.

Bushels are here calculated without *cones*.

1 bushel = 2150.42 cubic inches of distilled water at 40° Fahrenheit. Its dimensions are 18½ inches diameter inside, 8 inches deep, and when heaped the cone must be 6 inches high, or = 2748 cubic inches.

The imperial gallon = 277.274 cubic inches.

MEASUREMENT OF SURFACE.

Sq. miles.	Sq. acres.	Sq. rods.	Sq. yards.	Sq. feet.	Sq. inches.
1.	640.	102400.	3097600.	27878400.	4014489600.
.001562	1.	160.	4840.	43560.	696960.
	0.00625	1.	30.25	272.25	39204.
		0.033	1.	9.	1296.
			0.111	1.	144.
				0.00694	1.

TABLE OF SQUARES AND CUBES

Of all numbers from 1 to 500.

No.	Squares.	Cubes.	No.	Squares.	Cubes.
1	1	1	50	25 00	125 000
2	4	8	51	26 01	132 651
3	9	27	52	27 04	140 608
4	16	64	53	28 09	148 877
5	25	125	54	29 16	157 464
6	36	216	55	30 25	166 375
7	49	343	56	31 36	175 616
8	64	512	57	32 49	185 193
9	81	729	58	33 64	195 112
10	1 00	1 000	59	34 81	205 379
11	1 21	1 331	60	36 00	216 000
12	1 44	1 728	61	37 21	226 981
13	1 69	2 197	62	38 44	238 328
14	1 96	2 744	63	39 69	250 047
15	2 25	3 375	64	40 96	262 144
16	2 56	4 096	65	42 25	274 625
17	2 89	4 913	66	43 56	287 496
18	3 24	5 832	67	44 89	300 763
19	3 61	6 859	68	46 24	314 432
20	4 00	8 000	69	47 61	328 509
21	4 41	9 261	70	49 00	343 000
22	4 84	10 648	71	50 41	357 911
23	5 29	12 167	72	51 84	373 248
24	5 76	13 824	73	53 29	389 017
25	6 25	15 625	74	54 76	405 224
26	6 76	17 576	75	56 25	421 875
27	7 29	19 683	76	57 76	438 976
28	7 84	21 952	77	59 29	456 533
29	8 41	24 389	78	60 84	474 552
30	9 00	27 000	79	62 41	493 039
31	9 61	29 791	80	64 00	512 000
32	10 24	32 768	81	65 61	531 441
33	10 89	35 937	82	67 24	551 368
34	11 56	39 304	83	68 89	571 787
35	12 25	42 875	84	70 56	592 704
36	12 96	46 656	85	72 25	614 125
37	13 69	50 653	86	73 96	636 056
38	14 44	54 872	87	75 69	658 503
39	15 21	59 319	88	77 44	681 472
40	16 00	64 000	89	79 21	704 969
41	16 81	68 921	90	81 00	729 000
42	17 64	74 088	91	82 81	753 571
43	18 49	79 507	92	84 64	778 688
44	19 36	85 184	93	86 49	804 357
45	20 25	91 125	94	88 36	830 584
46	21 16	97 336	95	90 25	857 375
47	22 09	103 823	96	92 16	884 736
48	23 04	110 592	97	94 09	912 673
49	24 01	117 649	98	96 04	941 192

TABLE OF SQUARES AND CUBES, ETC.

No.	Squares.	Cubes.	No.	Squares.	Cubes.
99	98 01	970 299	156	2 43 36	3 796 416
100	1 00 00	1 000 000	157	2 46 49	3 869 893
101	1 02 01	1 030 301	158	2 49 64	3 944 312
102	1 04 04	1 061 208	159	2 52 81	4 019 679
103	1 06 09	1 092 727	160	2 56 00	4 096 000
104	1 08 16	1 124 864	161	2 59 21	4 173 281
105	1 10 25	1 157 625	162	2 62 44	4 251 528
106	1 12 36	1 191 016	163	2 65 69	4 330 747
107	1 14 49	1 225 043	164	2 68 96	4 410 944
108	1 16 64	1 259 712	165	2 72 25	4 492 125
109	1 18 81	1 295 029	166	2 75 56	4 574 296
110	1 21 00	1 331 000	167	2 78 89	4 657 463
111	1 23 21	1 367 631	168	2 82 24	4 741 632
112	1 25 44	1 404 928	169	2 85 61	4 826 809
113	1 27 69	1 442 897	170	2 89 00	4 913 000
114	1 29 96	1 481 544	171	2 92 41	5 000 211
115	1 32 25	1 520 875	172	2 95 84	5 088 448
116	1 34 56	1 560 896	173	2 99 29	5 177 717
117	1 36 89	1 601 613	174	3 02 76	5 268 024
118	1 39 24	1 643 032	175	3 06 25	5 359 375
119	1 41 61	1 685 159	176	3 09 76	5 451 776
120	1 44 00	1 728 000	177	3 13 29	5 545 233
121	1 46 41	1 771 561	178	3 16 84	5 639 752
122	1 48 84	1 815 848	179	3 20 41	5 735 339
123	1 51 29	1 860 867	180	3 24 00	5 832 000
124	1 53 76	1 906 624	181	3 27 61	5 929 741
125	1 56 25	1 953 125	182	3 31 24	6 028 568
126	1 58 76	2 000 376	183	3 34 89	6 128 487
127	1 61 29	2 048 383	184	3 38 56	6 229 504
128	1 63 84	2 097 152	185	3 42 25	6 331 625
129	1 66 41	2 146 689	186	3 45 96	6 434 856
130	1 69 00	2 197 000	187	3 49 69	6 539 203
131	1 71 61	2 248 091	188	3 53 44	6 644 672
132	1 74 24	2 299 968	189	3 57 21	6 751 269
133	1 76 89	2 352 637	190	3 61 00	6 859 000
134	1 79 56	2 406 104	191	3 64 81	6 967 871
135	1 82 25	2 460 375	192	3 68 64	7 077 888
136	1 84 96	2 515 456	193	3 72 49	7 189 057
137	1 87 69	2 571 353	194	3 76 36	7 301 384
138	1 90 44	2 628 072	195	3 80 25	7 414 875
139	1 93 21	2 685 619	196	3 84 16	7 529 536
140	1 96 00	2 744 000	197	3 88 09	7 645 373
141	1 98 81	2 803 221	198	3 92 04	7 762 392
142	2 01 64	2 863 288	199	3 96 01	7 880 599
143	2 04 49	2 924 207	200	4 00 00	8 000 000
144	2 07 36	2 985 984	201	4 04 01	8 120 601
145	2 10 25	3 048 625	202	4 08 04	8 242 408
146	2 13 16	3 112 136	203	4 12 09	8 365 427
147	2 16 09	3 176 523	204	4 16 16	8 489 664
148	2 19 04	3 241 792	205	4 20 25	8 615 125
149	2 22 01	3 307 949	206	4 24 36	8 741 816
150	2 25 00	3 375 000	207	4 28 49	8 869 743
151	2 28 01	3 442 951	208	4 32 64	8 998 912
152	2 31 04	3 511 808	209	4 36 81	9 129 329
153	2 34 09	3 581 577	210	4 41 00	9 261 000
154	2 37 16	3 652 264	211	4 45 21	9 393 931
155	2 40 25	3 723 875	212	4 49 44	9 528 128

TABLE OF SQUARES AND CUBES, ETC.

No.	Squares.	Cubes.	No.	Squares.	Cubes.
213	4 53 69	9 663 597	270	7 29 00	19 683 000
214	4 57 96	9 800 344	271	7 34 41	19 902 511
215	4 62 25	9 938 375	272	7 39 84	20 123 648
216	4 66 56	10 077 696	273	7 45 29	20 346 417
217	4 70 89	10 218 313	274	7 50 76	20 570 824
218	4 75 24	10 360 232	275	7 56 25	20 796 875
219	4 79 61	10 503 459	276	7 61 76	21 024 576
220	4 84 00	10 648 000	277	7 67 29	21 253 933
221	4 88 41	10 793 861	278	7 72 84	21 484 952
222	4 92 84	10 941 048	279	7 78 41	21 717 639
223	4 97 29	11 089 567	280	7 84 00	21 952 000
224	5 01 76	11 239 424	281	7 89 61	22 188 041
225	5 06 25	11 390 625	282	7 95 24	22 425 768
226	5 10 76	11 543 176	283	8 00 89	22 665 187
227	5 15 29	11 697 083	284	8 06 56	22 906 304
228	5 19 84	11 852 352	285	8 12 25	23 149 125
229	5 24 41	12 008 989	286	8 17 96	23 393 656
230	5 29 00	12 167 000	287	8 23 69	23 639 903
231	5 33 61	12 326 391	288	8 29 44	23 887 872
232	5 38 24	12 487 168	289	8 35 21	24 137 569
233	5 42 89	12 649 337	290	8 41 00	24 389 000
234	5 47 56	12 812 904	291	8 46 81	24 642 171
235	5 52 25	12 977 875	292	8 52 64	24 897 088
236	5 56 96	13 144 256	293	8 58 49	25 153 757
237	5 61 69	13 312 053	294	8 64 36	25 412 184
238	5 66 44	13 481 272	295	8 70 25	25 672 375
239	5 71 21	13 651 919	296	8 76 16	25 934 336
240	5 76 00	13 824 000	297	8 82 09	26 198 073
241	5 80 81	13 997 521	298	8 88 04	26 463 592
242	5 85 64	14 172 488	299	8 94 01	26 730 899
243	5 90 49	14 348 907	300	9 00 00	27 000 000
244	5 95 36	14 526 784	301	9 06 01	27 270 901
245	6 00 25	14 706 125	302	9 12 04	27 543 608
246	6 05 16	14 886 936	303	9 18 09	27 818 127
247	6 10 09	15 069 223	304	9 24 16	28 094 464
248	6 15 04	15 252 992	305	9 30 25	28 372 625
249	6 20 01	15 438 249	306	9 36 36	28 652 616
250	6 25 00	15 625 000	307	9 42 49	28 934 443
251	6 30 01	15 813 251	308	9 48 64	29 218 112
252	6 35 04	16 003 008	309	9 54 81	29 503 629
253	6 40 09	16 194 277	310	9 61 00	29 791 000
254	6 45 16	16 387 064	311	9 67 21	30 080 231
255	6 50 25	16 581 375	312	9 73 44	30 371 328
256	6 55 36	16 777 216	313	9 79 69	30 664 297
257	6 60 49	16 974 593	314	9 85 96	30 959 144
258	6 65 64	17 173 512	315	9 92 25	31 255 875
259	6 70 81	17 373 979	316	9 98 56	31 554 496
260	6 76 00	17 576 000	317	10 04 89	31 855 013
261	6 81 21	17 779 581	318	10 11 24	32 157 432
262	6 86 44	17 984 728	319	10 17 61	32 461 759
263	6 91 69	18 191 447	320	10 24 00	32 768 000
264	6 96 96	18 399 744	321	10 30 41	33 076 161
265	7 02 25	18 609 625	322	10 36 84	33 386 248
266	7 07 56	18 821 096	323	10 43 29	33 698 267
267	7 12 89	19 034 163	324	10 49 76	34 012 224
268	7 18 24	19 248 832	325	10 56 25	34 328 125
269	7 23 61	19 465 109	326	10 62 76	34 645 976

TABLE OF SQUARES AND CUBES, ETC.

No.	Squares.	Cubes.	No.	Squares.	Cubes.
327	10 69 29	34 965 783	384	14 74 56	56 623 104
328	10 75 84	35 287 552	385	14 82 25	56 066 625
329	10 82 41	35 611 289	386	14 89 96	57 512 456
330	10 89 00	35 937 000	387	14 97 69	57 960 603
331	10 95 61	36 264 691	388	15 05 44	58 411 072
332	11 02 24	36 594 368	389	15 13 21	58 863 869
333	11 08 89	36 926 037	390	15 21 00	59 319 000
334	11 15 56	37 259 704	391	15 28 81	59 776 471
335	11 22 25	37 595 375	392	15 36 64	60 236 288
336	11 28 96	37 933 056	393	15 44 49	60 698 457
337	11 35 69	38 272 753	394	15 52 36	61 162 984
338	11 42 44	38 614 472	395	15 60 25	61 629 875
339	11 49 21	38 958 219	396	15 68 16	62 099 136
340	11 56 00	39 304 000	397	15 76 09	62 570 773
341	11 62 81	39 651 821	398	15 84 04	63 044 792
342	11 69 64	40 001 688	399	15 92 01	63 521 199
343	11 76 49	40 353 607	400	16 00 00	64 000 000
344	11 83 36	40 707 584	401	16 08 01	64 481 201
345	11 90 25	41 063 625	402	16 16 04	64 964 808
346	11 97 16	41 421 736	403	16 24 09	65 450 827
347	12 04 09	41 781 923	404	16 32 16	65 939 264
348	12 11 04	42 144 192	405	16 40 25	66 430 125
349	12 18 01	42 508 549	406	16 48 36	66 923 416
350	12 25 00	42 875 000	407	16 56 49	67 419 143
351	12 32 01	43 243 551	408	16 64 64	67 917 312
352	12 39 04	43 614 208	409	16 72 81	68 417 929
353	12 46 09	43 986 977	410	16 81 00	68 921 000
354	12 53 16	44 361 864	411	16 89 21	69 426 531
355	12 60 25	44 738 875	412	16 97 44	69 934 528
356	12 67 36	45 118 016	413	17 05 69	70 444 997
357	12 74 49	45 499 293	414	17 13 96	70 957 944
358	12 81 64	45 882 712	415	17 22 25	71 473 375
359	12 88 81	46 268 279	416	17 30 56	71 991 296
360	12 96 00	46 656 000	417	17 38 89	72 511 713
361	13 03 21	47 045 881	418	17 47 24	73 034 632
362	13 10 44	47 437 928	419	17 55 61	73 560 059
363	13 17 69	47 832 147	420	17 64 00	74 088 000
364	13 24 96	48 228 544	421	17 72 41	74 618 461
365	13 32 25	48 627 125	422	17 80 84	75 151 448
366	13 39 56	49 027 896	423	17 89 29	75 686 967
367	13 46 89	49 430 863	424	17 97 76	76 225 024
368	13 54 24	49 836 032	425	18 06 25	76 765 625
369	13 61 61	50 243 409	426	18 14 76	77 308 776
370	13 69 00	50 653 000	427	18 23 29	77 854 483
371	13 76 41	51 064 811	428	18 31 84	78 402 752
372	13 83 84	51 478 848	429	18 40 41	78 953 589
373	13 91 29	51 895 117	430	18 49 00	79 507 000
374	13 98 76	52 313 624	431	18 57 61	80 062 991
375	14 06 25	52 734 375	432	18 66 24	80 621 568
376	14 13 76	53 157 376	433	18 74 89	81 182 737
377	14 21 29	53 582 633	434	18 83 56	81 746 504
378	14 28 84	54 010 152	435	18 92 25	82 312 875
379	14 36 41	54 439 939	436	19 00 96	82 881 856
380	14 44 00	54 872 000	437	19 09 69	83 453 453
381	14 51 61	55 306 341	438	19 18 44	84 027 672
382	14 59 24	55 742 968	439	19 27 21	84 604 519
383	14 66 89	56 181 887	440	19 36 00	85 184 000

TABLE OF SQUARES AND CUBES, ETC.

No.	Squares.	Cubes.	No.	Squares.	Cubes.
441	19 44 81	85 766 121	471	22 18 41	104 487 111
442	19 53 64	86 350 888	472	22 27 84	105 154 048
443	19 62 49	86 938 307	473	22 37 29	105 823 817
444	19 71 36	87 528 384	474	22 46 76	106 496 424
445	19 80 25	88 121 125	475	22 56 25	107 171 875
446	19 89 16	88 716 536	476	22 65 76	107 850 176
447	19 98 09	89 314 623	477	22 75 29	108 531 333
448	20 07 04	89 915 392	478	22 84 84	109 215 352
449	20 16 01	90 518 849	479	22 94 41	109 902 239
450	20 25 00	91 125 000	480	23 04 00	110 592 000
451	20 34 01	91 733 751	481	23 13 61	111 284 641
452	20 43 04	92 345 408	482	23 23 24	111 980 168
453	20 52 09	92 959 677	483	23 32 89	112 678 587
454	20 61 16	93 576 664	484	23 42 56	113 379 904
455	20 70 25	94 196 375	485	23 52 25	114 084 125
456	20 79 36	94 818 816	486	23 61 96	114 791 256
457	20 88 49	95 443 993	487	23 71 69	115 501 303
458	20 97 64	96 071 912	488	23 81 44	116 214 272
459	21 06 81	96 702 579	489	23 91 21	116 930 169
460	21 16 00	97 336 000	490	24 01 00	117 649 000
461	21 25 21	97 972 181	491	24 10 81	118 370 771
462	21 34 44	98 611 128	492	24 20 64	119 095 488
463	21 43 69	99 252 847	493	24 30 49	119 823 157
464	21 52 96	99 897 344	494	24 40 36	120 553 784
465	21 62 25	100 554 625	495	24 50 25	121 287 375
466	21 71 56	101 194 696	496	24 60 16	122 023 936
467	21 80 89	101 847 563	497	24 70 09	122 763 473
468	21 90 24	102 503 232	498	24 80 04	123 505 992
469	21 99 61	103 161 709	499	24 90 01	124 251 499
470	22 09 00	103 823 000	500	25 00 00	125 000 000

LENGTH OF CIRCULAR ARC.

Huygen's approximation.

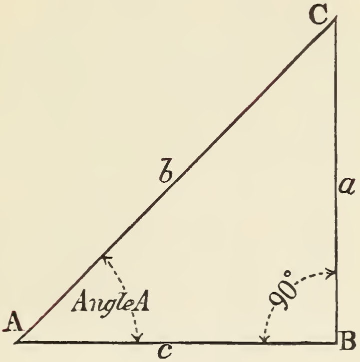
Huygen's approximation to length of a circular arc:
A = chord of any circular arc.
B = chord of half that arc.
R = radius of the circular arc.
L = length of the circular arc.
Then

$$L = \frac{8\,B - A}{3}$$

or, as it is usually written,

$$L = 2\,B + \tfrac{1}{3}\,(2\,B - A).$$

TRIGONOMETRICAL FUNCTIONS.



$\frac{a}{b} = \text{sine angle A.}$	$\frac{1}{\text{sine A}} = \frac{b}{a} = \text{cosecant angle A.}$
$\frac{c}{b} = \text{cosine “ A.}$	$\frac{1}{\text{cosine A}} = \frac{b}{c} = \text{secant “ A.}$
$\frac{a}{c} = \text{tangent “ A.}$	$\frac{1}{\text{tangent A}} \doteq \frac{c}{a} = \text{cotangent “ A.}$

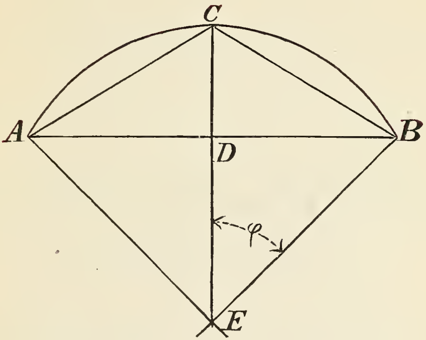
Therefore,

$a = b \times \text{sine A.}$	$b = c \times \text{secant A.}$
$b = a \times \text{cosecant A.}$	$a = c \times \text{tangent A.}$
$c = b \times \text{cosine A.}$	$c = a \times \text{cotangent A.}$

NATURAL SINES, ETC.

Deg.	Sine.	Cover.	Cosecant	Tangent.	Cotang.	Secant.	Versin.	Cosin.	Deg.
0	0.0	1.00000	Infinite.	0.0	Infinite.	1.00000	0.0	1.00000	90
1	0.01745	0.98254	57.2986	0.01745	57.2899	1.00015	0.0001	0.99984	89
2	0.03489	0.96510	28.6537	0.03492	28.6362	1.00060	0.0006	0.99939	88
3	0.05233	0.94766	19.1073	0.05240	19.0811	1.00137	0.0013	0.99862	87
4	0.06975	0.93024	14.3355	0.06992	14.3006	1.00244	0.0024	0.99756	86
5	0.08715	0.91284	11.4737	0.08748	11.4300	1.00381	0.0038	0.99619	85
6	0.10452	0.89547	9.5667	0.10510	9.5143	1.00550	0.0054	0.99452	84
7	0.12186	0.87813	8.2055	0.12278	8.1443	1.00750	0.0074	0.99254	83
8	0.13917	0.86082	7.1852	0.14054	7.1153	1.00982	0.0097	0.99026	82
9	0.15643	0.84356	6.3924	0.15838	6.3137	1.01246	0.0123	0.98768	81
10	0.17364	0.82635	5.7587	0.17632	5.6712	1.01542	0.0151	0.98480	80
11	0.19080	0.80919	5.2408	0.19438	5.1445	1.01871	0.0183	0.98162	79
12	0.20791	0.79208	4.8097	0.21255	4.7046	1.02234	0.0218	0.97814	78
13	0.22495	0.77504	4.4454	0.23086	4.3314	1.02630	0.0256	0.97437	77
14	0.24192	0.75807	4.1335	0.24932	4.0107	1.03061	0.0297	0.97029	76
15	0.25881	0.74118	3.8637	0.26794	3.7320	1.03527	0.0340	0.96592	75
16	0.27563	0.72436	3.6279	0.28674	3.4874	1.04029	0.0387	0.96126	74
17	0.29237	0.70762	3.4203	0.30573	3.2708	1.04569	0.0436	0.95630	73
18	0.30901	0.69098	3.2360	0.32491	3.0776	1.05146	0.0489	0.95105	72
19	0.32556	0.67443	3.0715	0.34432	2.9042	1.05762	0.0544	0.94551	71
20	0.34202	0.65797	2.9238	0.36397	2.7474	1.06417	0.0603	0.93969	70
21	0.35836	0.64163	2.7904	0.38386	2.6050	1.07114	0.0664	0.93358	69
22	0.37460	0.62539	2.6694	0.40402	2.4750	1.07853	0.0728	0.92718	68
23	0.39073	0.60926	2.5593	0.42447	2.3558	1.08636	0.0794	0.92050	67
24	0.40673	0.59326	2.4585	0.44522	2.2460	1.09463	0.0864	0.91354	66
25	0.42261	0.57738	2.3662	0.46630	2.1445	1.10337	0.0936	0.90630	65
26	0.43837	0.56162	2.2811	0.48773	2.0503	1.11260	0.1012	0.89879	64
27	0.45399	0.54600	2.2026	0.50952	1.9626	1.12232	0.1089	0.89100	63
28	0.46947	0.53052	2.1300	0.53170	1.8807	1.13257	0.1170	0.88294	62
29	0.48480	0.51519	2.0626	0.55430	1.8040	1.14335	0.1253	0.87461	61
30	0.50000	0.50000	2.0000	0.57735	1.7320	1.15470	0.1339	0.86602	60
31	0.51503	0.48496	1.9416	0.60086	1.6642	1.16663	0.1428	0.85716	59
32	0.52991	0.47008	1.8870	0.62486	1.6003	1.17917	0.1519	0.84804	58
33	0.54463	0.45536	1.8360	0.64940	1.5398	1.19236	0.1613	0.83867	57
34	0.55919	0.44080	1.7882	0.67450	1.4825	1.20621	0.1709	0.82903	56
35	0.57357	0.42642	1.7434	0.70020	1.4281	1.22077	0.1808	0.81915	55
36	0.58778	0.41221	1.7013	0.72654	1.3763	1.23606	0.1909	0.80901	54
37	0.60181	0.39818	1.6616	0.75355	1.3270	1.25213	0.2013	0.79863	53
38	0.61566	0.38433	1.6242	0.78128	1.2799	1.26901	0.2119	0.78801	52
39	0.62932	0.37067	1.5890	0.80978	1.2348	1.28675	0.2228	0.77714	51
40	0.64278	0.35721	1.5557	0.83909	1.1917	1.30540	0.2339	0.76604	50
41	0.65605	0.34394	1.5242	0.86928	1.1503	1.32501	0.2452	0.75470	49
42	0.66913	0.33086	1.4944	0.90040	1.1106	1.34563	0.2568	0.74314	48
43	0.68199	0.31800	1.4662	0.93251	1.0723	1.36732	0.2686	0.73135	47
44	0.69465	0.30534	1.4395	0.96568	1.0355	1.39016	0.2806	0.71933	46
45	0.70710	0.29289	1.4142	1.00000	1.0000	1.41421	0.2928	0.70710	45
	Cosin.	Versin.	Secant.	Cotang.	Tangent.	Cosecant	Cover.	Sine.	

PROPERTIES OF CIRCULAR ARCS.



C D = v.
A B = c.
C D = v = r (1 — cos. ϕ).

$$\text{Sin } \phi = \frac{\frac{1}{2}c}{r}$$

Given, chord A B = c, and ver. sine C D = v, required radius r.

$$\frac{AB}{2} = AD = DB$$

then

$$\frac{\overline{AD}^2 + \overline{DC}^2}{2\,DC} = CE$$

i.e.,

$$\frac{c^2 + 4\,\overline{v}^2}{8\,v}$$

Given, chord A B and radius C E, to find rise C D.

$$CE - \sqrt{\overline{CE}^2 - \overline{AD}^2} = CD$$

i.e.,

$$r - \sqrt{r^2 - \frac{c^2}{4}}$$

Given, the radius and rise or vers. sine, to find the chord A B.

$$AD = \sqrt{\overline{CE}^2 - (CE - CD)^2}$$

i.e.,

$$c = 2\sqrt{2\,vr - v^2}$$

TABLE OF PROPORTIONS OF THE CIRCLE AND ITS EQUAL.

The diameter of any circle $\times$ 3.1416 $=$ the circumference.

The circumference of any circle $\times \left(\frac{1}{3.1416} = 0.31831\right)$ $=$ the diameter.

The square of the diameter $\times \left(\frac{3.1416}{4} = 0.7854\right) =$ the area.

The square of the circumference $\times \left(\frac{0.7854}{3.1416^2} = 0.07958\right)$ $=$ the area.

The diameter of a circle $\times$ ($\sqrt{0.7854} = 0.8862$) $=$ side of equal square.

The circumference of a circle $\times$ ($\sqrt{0.07958} = 0.2821$) $=$ side of equal square.

The side of any square $\times \left(\frac{1}{0.8862} = 1.1284\right) =$ diameter of equal circle.

The side of any square $\times \left(\frac{1}{0.2821} = 3.545\right) =$ circumference of equal circle.

Square of side $\times \left(\frac{1}{0.7854} = 1.27324366\right) =$ square of diameter of equal circle = so-called round inches.

Round inches $\times \left(\frac{0.7854}{144.} = 0.0546\right) =$ square feet.

Square of diameter of equal circle $\times$ 0.7854 = square of side.

Area of segment of circle = area of sector of equal radius, less area of triangle.

Area of parabola = base $\times \frac{2}{3}$ height.

Area of ellipse = longest diameter $\times$ shortest diameter $\times$ .7854.

Area of any regular polygon = sum of its sides $\times$ perpendicular from its centre to one of its sides, divided by 2.

Surface of cylinder = area of both ends + length $\times$ circumference.

Surface of segment = height of segment $\times$ whole circumference of sphere of which it is a part.

Cubic contents of a cylinder = area of one end $\times$ length.

AREAS OF CIRCLES.

Advancing by eighths.

Diam.	AREAS. .0	⅛	¼	⅜	½	⅝	¾	⅞
0	0.0	0.0122	0.0490	0.1104	0.1963	0.3068	0.4417	0.6013
1	0.7854	0.9940	1.227	1.484	1.767	2.073	2.405	2.761
2	3.1416	3.546	3.976	4.430	4.908	5.411	5.939	6.491
3	7.068	7.669	8.295	8.946	9.621	10.32	11.04	11.79
4	12.56	13.36	14.18	15.03	15.90	16.80	17.72	18.66
5	19.63	20.62	21.64	22.69	23.75	24.85	25.96	27.10
6	28.27	29.46	30.67	31.91	33.18	34.47	35.78	37.12
7	38.48	39.87	41.28	42.71	44.17	45.66	47.17	48.70
8	50.26	51.84	53.45	55.08	56.74	58.42	60.13	61.86
9	63.61	65.39	67.20	69.02	70.88	72.75	74.69	76.58
10	78.54	80.51	82.51	84.54	86.59	88.66	90.76	92.88
11	95.03	97.20	99.40	101.6	103.8	106.1	108.4	110.7
12	113.0	115.4	117.8	120.2	122.7	125.1	127.6	130.1
13	132.7	135.2	137.8	140.5	143.1	145.8	148.4	151.2
14	153.9	156.6	159.4	162.2	165.1	167.9	170.8	173.7
15	176.7	179.6	182.6	185.6	188.6	191.7	194.8	197.9
16	201.0	204.2	207.3	210.5	213.8	217.0	220.3	223.6
17	226.9	230.3	233.7	237.1	240.5	243.9	247.4	250.9
18	254.4	258.0	261.5	265.1	268.8	272.4	276.1	279.8
19	283.5	287.2	291.0	294.8	298.6	302.4	306.3	310.2
20	314.1	318.1	322.0	326.0	330.0	334.1	338.1	342.2
21	346.3	350.4	354.6	358.8	363.0	367.2	371.5	375.8
22	380.1	384.4	388.8	393.2	397.6	402.0	406.4	410.9
23	415.4	420.0	424.5	429.1	433.7	438.3	443.0	447.6
24	452.3	457.1	461.8	466.6	471.4	476.2	481.1	485.9
25	490.8	495.7	500.7	505.7	510.7	515.7	520.7	525.8
26	530.9	536.0	541.1	546.3	551.5	556.7	562.0	567.2
27	572.5	577.8	583.2	588.5	593.9	599.3	604.8	610.2
28	615.7	621.2	626.7	632.3	637.9	643.5	649.1	654.8
29	660.5	666.2	671.9	677.7	683.4	689.2	695.1	700.9
30	706.8	712.7	718.6	724.6	730.6	736.6	742.6	748.6
31	754.8	760.9	767.0	773.1	779.3	785.5	791.7	798.0
32	804.3	810.6	816.9	823.2	829.6	836.0	842.4	848.8
33	855.3	861.8	868.3	874.9	881.4	888.0	894.6	901.3
34	907.9	914.7	921.3	928.1	934.8	941.6	948.4	955.3
35	962.1	969.0	975.9	982.8	989.8	996.8	1003.8	1010.8
36	1017.9	1025.0	1032.1	1039.2	1046.3	1053.5	1060.7	1068.0
37	1075.2	1082.5	1089.8	1097.1	1104.5	1111.8	1119.2	1126.7
38	1134.1	1141.6	1149.1	1156.6	1164.2	1171.7	1179.3	1186.9
39	1194.6	1202.3	1210.0	1217.7	1225.4	1233.2	1241.0	1248.8
40	1256.6	1264.5	1272.4	1280.3	1288.2	1296.2	1304.2	1312.2
41	1320.3	1328.3	1336.4	1344.5	1352.7	1360.8	1369.0	1377.2
42	1385.4	1393.7	1402.0	1410.3	1418.6	1427.0	1435.4	1443.8
43	1452.2	1460.7	1469.1	1477.6	1486.2	1494.7	1503.3	1511.9
44	1520.5	1529.2	1537.9	1546.6	1555.3	1564.0	1572.8	1581.6
45	1590.4	1599.3	1608.2	1617.0	1626.0	1634.9	1643.9	1652.9

CIRCUMFERENCES OF CIRCLES.

Advancing by eighths.

	CIRCUMFERENCES.							
Diam.	.0	.1/8	.1/4	.3/8	.1/2	.5/8	.3/4	.7/8
0	0.0	0.3927	0.7854	1.178	1.570	1.963	2.356	2.748
1	3.141	3.534	3.927	4.319	4.712	5.105	5.497	5.890
2	6.283	6.675	7.068	7.461	7.854	8.246	8.639	9.032
3	9.424	9.817	10.21	10.60	10.99	11.38	11.78	12.17
4	12.56	12.95	13.35	13.74	14.13	14.52	14.92	15.31
5	15.70	16.10	16.49	16.88	17.27	17.67	18.06	18.45
6	18.84	19.24	19.63	20.02	20.42	20.81	21.20	21.59
7	21.99	22.38	22.77	23.16	23.56	23.95	24.34	24.74
8	25.13	25.52	25.91	26.31	26.70	27.09	27.48	27.88
9	28.27	28.66	29.05	29.45	29.84	30.23	30.63	31.02
10	31.41	31.80	32.20	32.59	32.98	33.37	33.77	34.16
11	34.55	34.95	35.34	35.73	36.12	36.52	36.91	37.30
12	37.69	38.09	38.48	38.87	39.27	39.66	40.05	40.44
13	40.84	41.23	41.62	42.01	42.41	42.80	43.19	43.58
14	43.98	44.37	44.76	45.16	45.55	45.94	46.33	46.73
15	47.12	47.51	47.90	48.30	48.69	49.08	49.48	49.87
16	50.26	50.65	51.05	51.44	51.83	52.22	52.62	53.01
17	53.40	53.79	54.19	54.58	54.97	55.37	55.76	56.15
18	56.54	56.94	57.33	57.72	58.11	58.51	58.90	59.29
19	59.69	60.08	60.47	60.86	61.26	61.65	62.04	62.43
20	62.83	63.22	63.61	64.01	64.40	64.79	65.18	65.58
21	65.97	66.36	66.75	67.15	67.54	67.93	68.32	68.72
22	69.11	69.50	69.90	70.29	70.68	71.07	71.47	71.86
23	72.25	72.64	73.04	73.43	73.82	74.22	74.61	75.00
24	75.39	75.79	76.18	76.57	76.96	77.36	77.75	78.14
25	78.54	78.93	79.32	79.71	80.10	80.50	80.89	81.28
26	81.68	82.07	82.46	82.85	83.25	83.64	84.03	84.43
27	84.82	85.21	85.60	86.00	86.39	86.78	87.17	87.57
28	87.96	88.35	88.75	89.14	89.53	89.92	90.32	90.71
29	91.10	91.49	91.89	92.28	92.67	93.06	93.46	93.85
30	94.24	94.64	95.03	95.42	95.81	96.21	96.60	96.99
31	97.39	97.78	98.17	98.57	98.96	99.35	99.75	100.14
32	100.53	100.92	101.32	101.71	102.10	102.49	102.89	103.29
33	103.67	104.07	104.46	104.85	105.24	105.64	106.03	106.42
34	106.81	107.21	107.60	107.99	108.39	108.78	109.17	109.56
35	109.96	110.35	110.74	111.13	111.53	111.92	112.31	112.71
36	113.10	113.49	113.88	114.28	114.67	115.06	115.45	115.85
37	116.24	116.63	117.02	117.42	117.81	118.20	118.60	118.99
38	119.38	119.77	120.17	120.56	120.95	121.34	121.74	122.13
39	122.52	122.92	123.31	123.70	124.09	124.49	124.88	125.27
40	125.66	126.06	126.45	126.84	127.24	127.63	128.02	128.41
41	128.81	129.20	129.59	129.98	130.38	130.77	131.16	131.55
42	131.95	132.34	132.73	133.13	133.52	133.91	134.30	134.70
43	135.09	135.48	135.87	136.27	136.66	137.05	137.45	137.84
44	138.23	138.62	139.02	139.41	139.80	140.19	140.59	140.98
45	141.37	141.76	142.16	142.55	142.94	143.34	143.73	144.12

CONSTANTS RELATING TO THE CIRCLE.

		Constant.	Log.
Circumference of circle $= \pi \times$ diam.			
Surface of sphere $= \pi$ (diam.)^2		3.14159	0.49715
Area of circle $= \pi \times$ (radius)^2			
Circumference of circle $= 2\pi \times$ radius .	2π	6.28318	0.79818
Area of circle $= \frac{1}{4}\pi \times$ (diam.)^2 . . .	$\frac{1}{4}\pi$	0.785398	$\bar{1}.89509$
Surface of sphere $= 4\pi \times$ (radius)^2 . .	4π	12.56637	1.09921
Volume of sphere $= \frac{1}{6}\pi \times$ (diam.)^3 .	$\frac{1}{6}\pi$	0.52359	$\bar{1}.71900$
Volume of sphere $= \frac{4}{3}\pi$ (radius)^3 . . .	$\frac{4}{3}\pi$	4.18879	0.62209
Square of π	π^2	9.86960	0.99430
Square root of π	$\sqrt{\pi}$	1.772454	0.24857
Cube root of π	$\sqrt[3]{\pi}$	1.46459	0.16572
360° expressed in seconds		1296000	6.11261
360° expressed in minutes		21600	4.33445
Arc equal radius expressed in seconds .		206264.8	5.31442
Arc equal radius expressed in minutes .		3437.747	3.53627
Arc equal radius expressed in degrees .	$\frac{180}{\pi}$	57.29578	1.75812
Length of arc, $1'' = \sin 1''$	$\sin 1''$	0.000004848	$\bar{6}.68557$
Length of arc, $1' = \sin 1'$	$\sin 1''$	0.0002909	$\bar{4}.46373$

CONSTANTS RELATING TO LOGARITHMIC SYSTEMS.

		Constant.	Log.
Base of Napierian system	l	2.7182818	0.43429
Modulus of Brigg's system	M	0.434294	$\bar{1}.63778$
Reciprocal of modulus	K	2.302585	0.36222

CONSTANTS RELATING TO GRAVITY.

	Constant.
Cubic inch of distilled water at 62° F., in grains	252.458
Cubic inch of distilled water at 60° F., in grains	252.500
Cubic inch of distilled water at 4° C., in grains	252.890
Cubic foot of distilled water at 60° F., in ounces av. . .	997.310
Cubic foot of distilled water at 60° F., in pounds av. . .	62.33184
Cubic inch of mercury at 32° F., in grains	3438.8
Cubic inch of mercury at 32° F., in pounds av.	0.49125
Seconds pendulum, in inches, at London	39.139
Seconds pendulum, in inches, at Pole	39.218
Seconds pendulum, in inches, at Latitude 45°	39.118
Seconds pendulum, in inches, at Equator	39.018
Gravity, in feet, at London	32.1908
Gravity, in feet, at Pole	32.2552
Gravity, in feet, at Latitude 45°	32.1736
Gravity, in feet, at Equator	32.0907

REDUCTION MULTIPLIERS.

FOR CONVERTING	Constant.
Barometric inches (32° F.) into pounds per square inch	0.49125
Barometric millimetres (32° F.) into kilogrammes per square centimetre	0.00136
Kilogrammes per square centimetre into pounds per square inch	14.22263
Foot-pounds into kilogrammetres	0.13825

HEAT.

THERMOMETERS.

To convert the degrees of different thermometers, from one into the other, use the following formula:

F stands for degrees of Fahrenheit, or 212°
C stands for degrees of Celsius, or 100°
R stands for degrees of Reaumur, or 80°
} boiling point.

$F = \frac{9R}{4} + 32$ and $F = \frac{9C}{5} + 32$ for degrees above freezing point.

$F = \frac{9R}{4} - 32$ and $F = \frac{9C}{5} - 32$ for degrees below freezing point.

$C = \frac{5(F - 32)}{9}$ and $R = \frac{4(F - 32)}{9}$ for degrees above freezing point.

$C = \frac{5(F + 32)}{9}$ and $R = \frac{4(F + 32)}{9}$ for degrees below freezing point.

Zero of Celsius or Reaumur is $= + 32°$ Fahrenheit.

Zero of Fahrenheit $= - 17.77°$ C. or $- 14.22°$ R.

1. How much is 8° Celsius above zero in Fahrenheit?

$$F = \frac{9 \times 8}{5} = \frac{72}{5} = 14.4 + 32 = 46.4° \text{ above.}$$

2. How much is 8° Celsius below zero in Fahrenheit?

$$F = \frac{9 \times 8}{5} = \frac{72}{5} = 14.4 - 32 = 17.6° \text{ above.}$$

In cases where the product is smaller than 32, it indicates that the degree is above zero of Fahrenheit. See Example 2.

3. How much is 19° Celsius below zero in Fahrenheit?

$$F = \frac{9 \times 19}{5} - 32 = 34.2 - 32 = 2.2° \text{ below Fahrenheit.}$$